pieces of short-leaf or loblolly pine might have grown under such conditions as to possess the density necessary to give them rating with first-class dense long-leaf pieces.

In other words, the Forest Service experiments showed that the strength of Southern yellow-pine timber is not dependent necessarily on its botanical species—a finding that was revolutionary in view of all precedent, which had specified long-leaf yellow pine for timbers required for exceptional uses.

### Early Proposals to Classify by Density

While the Forest Service was engaged upon this experimental work, the American Society for Testing Materials was active in the same direction. The society began a series of investigations in 1909, and a committee of the society at that time pointed out that in practice the separation of the various species of yellow-pine timbers after manufacture is extremely difficult, and in many cases practically impossible. In other words, it was easy enough to specify that all timbers should be manufactured from strictly long-leaf Southern yellow pine, but it was in many cases impossible to determine whether the timbers actually received were really long-leaf or some other species. The committee also suggested that, whether they were long-leaf or not, the difference in strength in the various pieces was probably due to a very large extent to the different percentages of wood fiber per cubic foot—that is, the different densities of the various pieces.

With this in mind the committee formulated a tentative rule, suggesting the advisability of dropping the botanical designations and classifying the Southern pines into two groups, the basis of which classification was to be the number of annual rings per inch, as counted on a cross-section

A suggestion was made by A. O. Elzner, representing the American Institute of Architects, that it might be just as well to drop the terms long-leaf, short-leaf and loblolly altogether and call all Southern yellow-pine timbers Southern yellow pine, to be divided into grades 1, 2 and 3, according to the number of annual rings per inch.

### Formulating the Density Rule

To shorten the story of a long and laborious investigation on the part of the Forest Service and of the American Society of Testing Materials, the former suggested a rule using the rate of growth, the percentage of summer wood and a distinct line of demarcation of spring and summer wood as the basis for the new classification.

The American Society for Testing Materials was put in possession of all these data; and after studying them thoroughly the committee recommended the adoption with only slight modification of the rule formulated by the Government. In this form the rule was unanimously adopted by the society in August of last year.

Awake to the necessity of such a rule and recognizing the value and dependability of the findings of the Forest Service and of the American Society for Testing Materials, the Southern Pine Association, an organization made up of manufacturers of Southern yellow-pine timbers, promptly indorsed the rule and adopted it.

The rule is characterized by shortness and simplicity:

**Dense Southern yellow pine shall show on either end an average of at least six annual rings per inch and at least one-third summer wood, or else the greater number of the rings shall show at least one-third summer wood, all as measured over the third, fourth and fifth inches on a radial line from the pith. Wide-ringed material excluded by this rule will be acceptable provided that the amount of summer wood as above measured shall be at least one-half.**

**The contrast in color between summer wood and spring wood shall be sharp, and the summer wood shall be dark in color, except in pieces having considerably above the minimum requirement for summer wood.**

**In cases where timbers do not contain the pith and it is impossible to locate it with any degree of accuracy the same inspection shall be made over 3 in. on an approximate radial line beginning at the edge nearest the pith in timbers over 3 in. in thickness and on the second inch (on the piece) nearest to the pith in timbers 3 in. or less in thickness.**

**In dimension material containing the pith, but not a 5-in. radial line, which is less than 2x8 in. in section or less than 8 in. in width, that does not show over 16 sq.in. on the cross-section, the inspection shall apply to the second inch from the pith. In larger material that does not show a 5-in. radial line the inspection shall apply to the 3 in. farthest from the pith.**

**Sound Southern yellow pine shall include pieces of Southern pine without any ring or summer-wood requirement.**

### Merits of the Density Rule

While the new rule may at first sight appear to be a somewhat radical departure from past standards, a careful study will show that such is not really the case. The density rule, when applied to a mixed lot of the various Southern pine timbers of the several botanical species, will include most of the pieces of the true botanical long-leaf pine in the grade hereafter to be known as "dense pine"; a smaller percentage of the denser pieces of loblolly, Cuban and short-leaf pine also will fall within the dense grade; and on the other hand, a small percentage of the more rapid-growing pieces of long-leaf will be excluded, as will also a great majority of pieces of loblolly, Cuban and short-leaf.

It should be clearly understood that the classes "dense pine" and the less desirable "sound pine" refer specifically to quality of density when considered from the structural or strength standpoint and that they replace the botanical terms hitherto used—long-leaf, short-leaf, loblolly, etc.

It should also be clearly understood that the usual specifications as to the percentage of heart and sap are in no way changed. In other words, where timber for long service is demanded, together with high strength qualities, dense pine should be specified with a minimum amount of sapwood, or what is known as heart timber.

A feature of the new rule that particularly recommends it is that it is easy of application. For the first time there is available for classifying structural timbers a rule which is based on actual measurement and which has nothing to do with catch-judgment or feeling. Numerous diagrams and illustrations in the Southern Pine Association's density-rule book show how simple is the method and how effectually disagreements are eliminated.

To promote the practice of standardizing high-grade dense timbers, the association just mentioned has adopted the policy of branding each individual timber. When this practice has been generally adopted, it will be a further guarantee of quality, serving to make assurance doubly sure to the builder having occasion to use yellow pine.

It seems to me that every architect and engineer in America should make it his business immediately to familiarize himself with this new grading rule and the method of its application. Since yellow pine has qualities that make it most desirable for certain uses, it is a matter for congratulation to manufacturers and users that the uncertainties have been eliminated in specifying the material in heavy construction. This is the more true because, as already stated, ample supplies of yellow pine are available, opinion to the contrary notwithstanding.

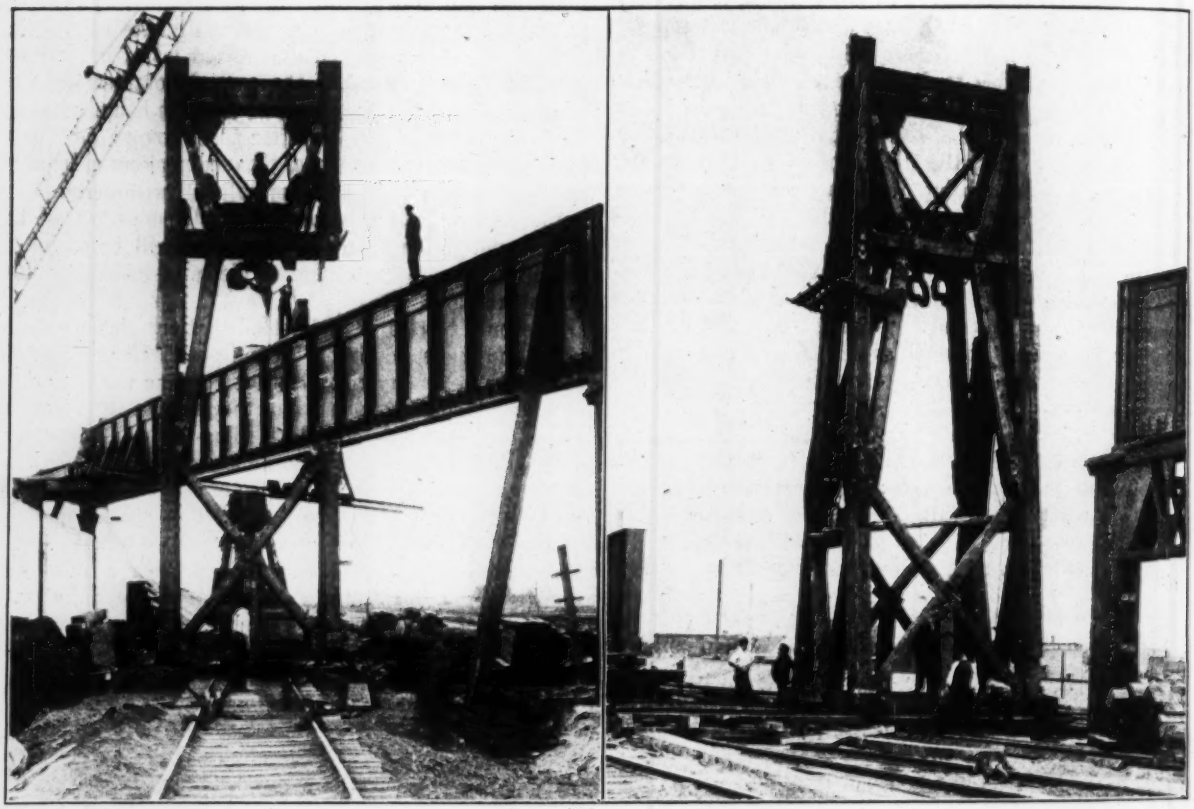
# Field and Office

## Erecting Long Plate Girders by Tower Traveler

To erect the record-breaking plate girders of the five-span Nickel Plate crossing over the Illinois Central R.R. tracks at 79th St., Chicago, required a plant and method capable of quick work, to avoid blocking the traffic on the Illinois Central tracks below. The bridge itself and the heavy girders composing its five spans were described and illustrated by drawings in *Engineering News* of Dec. 9, 1915, p. 1105. The heaviest girders weigh 130½ tons and are 125 ft. 8 11/16 in. long; the longest girders are 131 ft. 9 in. over all.

All girders were received on cars on the Illinois Central tracks and were hoisted up to place by a four-leg tower that straddled the girders being lifted. Removable leg sections in the rear bent of the tower provided for disengaging it from a girder just erected and moving it forward to the next. The tower was slid from girder to girder on the track rails. In sliding it over to the next span, transverse rails laid on the track rails served as bed. The group of views herewith shows some of the typical stages in the erection work and illustrates the general procedure clearly.

Figs. 1 and 2 show the tower and the method of moving. How it was used can be seen best from the large view, Fig. 4. Straddling one of the Illinois Central tracks, the tower picked up a girder brought in on cars on this track. A set of hooks attached to a loading beam suspended in the tower engaged the girder at the middle (the girders sometimes had to be balanced by iron piled on one end). Then the girder was hoisted to the level of its final position, Fig. 3, swung around to the alignment of the bridge and set on its supports, Fig. 4. The first girder, Fig. 1, when set, was strutted at both ends, besides being held by the attachment of floor-beams at the abutment.



FIGS. 1 AND 2. THE A-FRAME AND TOWER USED FOR ERECTING THE NICKEL-PLATE GIRDERS AT 79TH ST., CHICAGO—METHOD OF MOVING ALONG AND ACROSS TRACKS

Fig. 1—First girder set on final bearings and braced by floor-beams and struts; lower panel of cross-bracing inserted in tower, and "gate" section of posts on one side removed; tower just being hauled forward to location of next girder. Fig. 2—Sliding the tower to the next span on rails laid over the Illinois Central R.R. rails

The condition of the tower in Fig. 1 is that required for moving forward along the track to the second girder. Crossbracing is inserted in the lower panel of the tower, and in the panel next above, the rear legs are removed, leaving the top frame of the tower cantilevered from the other pair of legs. Thus the tower can be slid forward clear of the girder it has just set and ready for reassembling to take hold of another girder.

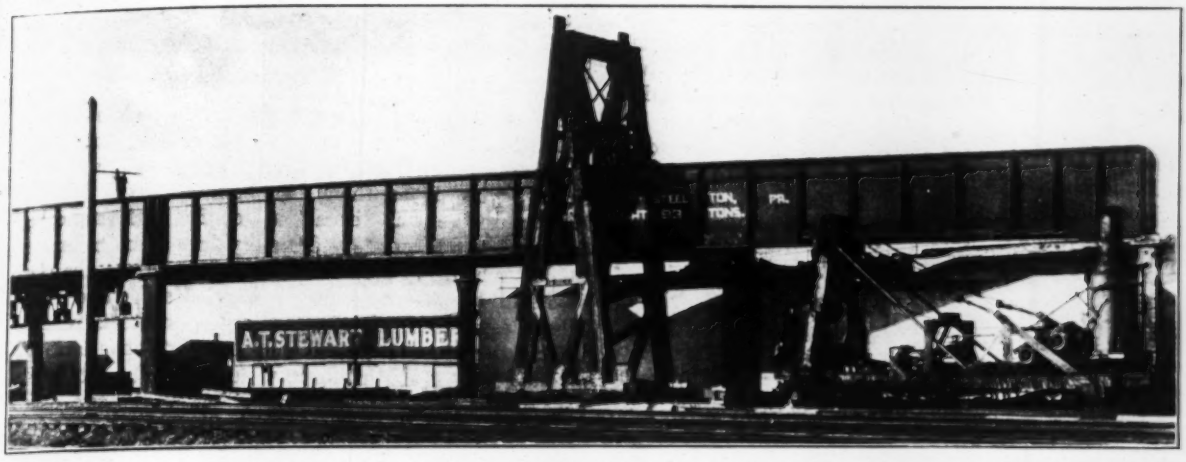
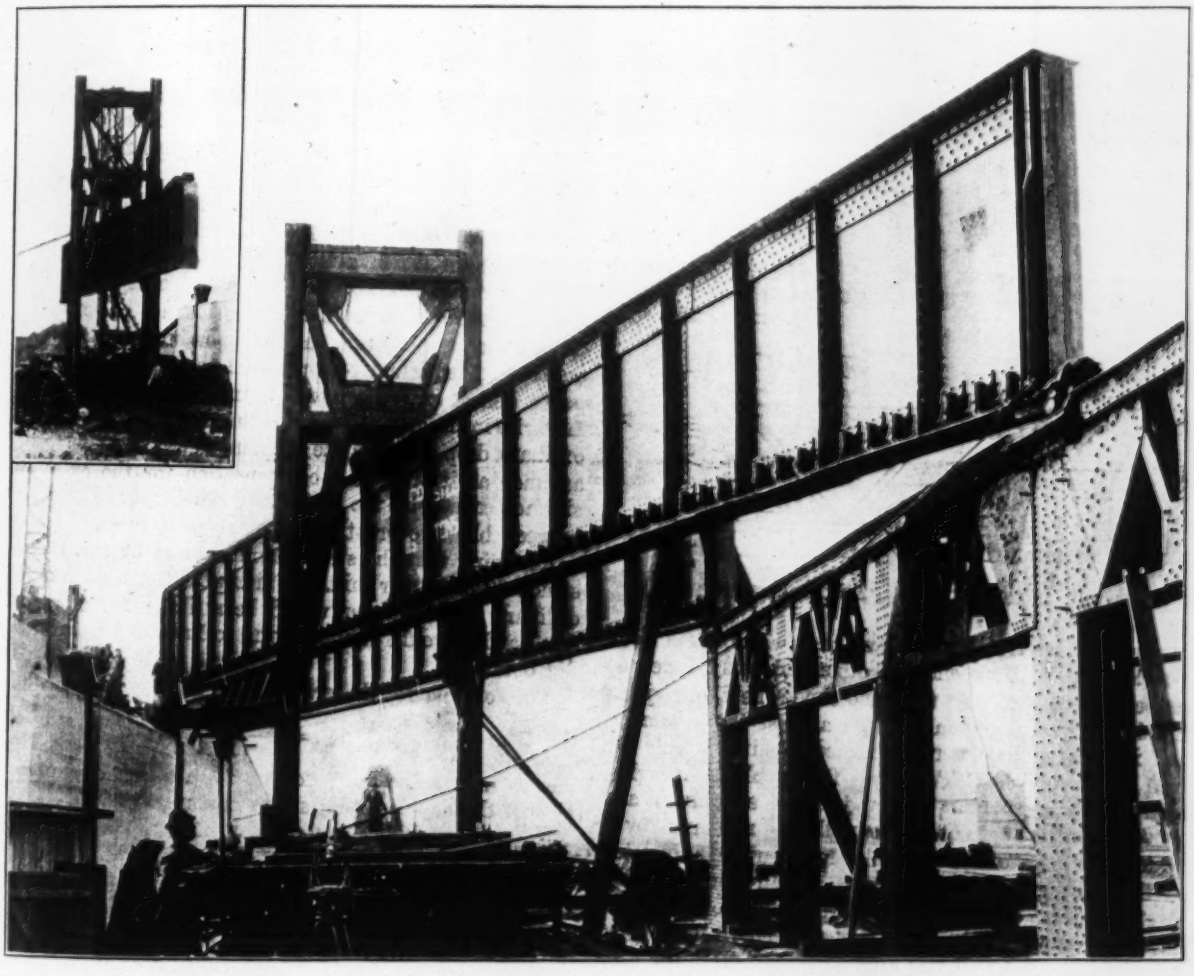



FIG. 5. LAST GIRDER OF BRIDGE SET IN PLACE OCT. 29, 1915

The tower had to be placed quite accurately, as the girder, when swung, could be moved only a trifle endwise and practically nothing laterally. This limitation meant also that the tower had to be set level. In spite of the care required in this work, the erection attained a speed of one girder every other day, after the gang got fairly under way. The first of the 15 girders was set Sept. 15, and the last on Oct. 29, 1915.

In preparing for the erection the field party located the bearings to the precise spacings of the girder shoes, as given by shop measurement after fabrication. As a result, no bearing missed connection by more than $\frac{1}{16}$ in.



FIGS. 3 AND 4. METHOD OF HOISTING AND SETTING GIRDERS WITH THE TOWER

Fig. 3, Sept. 15, 1915, shows the first girder of the bridge being lifted from cars, ready to swing around into place. In Fig. 4 the second girder is just being landed on its bearings; compare with Fig. 1 to note condition of tower for moving and for hoisting

It must be remembered that the bridge is on a very sharp skew (about 20° to Illinois Central tracks), which increases the difficulties of accurate setting. The two girders meeting on a bent have a joint base-plate, with integral half-pin seats for notches in the girder shoes.

The bridge was erected for the New York, Chicago & St. Louis R.R., of which George H. Tinker is Bridge Engineer; A. C. Harvey was Field Engineer. The work was done by the Pennsylvania Steel Co., Steelton, Penn., under J. F. Ritter, Superintendent of Erection.

## Thrust of Steel Arch Adjusted by End Jacking

A railway street crossing over Markt St., Berlin, Germany, which has its girders and columns formed in one piece, acting as an arch structure, was adjusted in the course of erection to precise theoretical action by horizontal jacking. R. Kramer describes this process in *Der Eisenbau* of September, 1915. In earlier cases of jacking two-hinged arches the force was applied at the crown. Jacking at the abutment is a novel procedure.

The bridge has a span of 80 ft. and comprises seven parallel girders or frames. Each girder was erected on

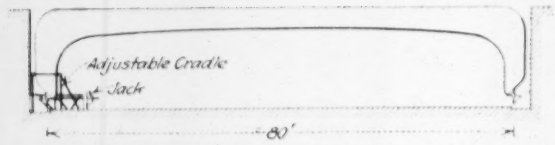



FIG. 1. SKETCH OF ARCH STREET CROSSING, CRADLE AND HORIZONTAL JACKING ARRANGEMENT

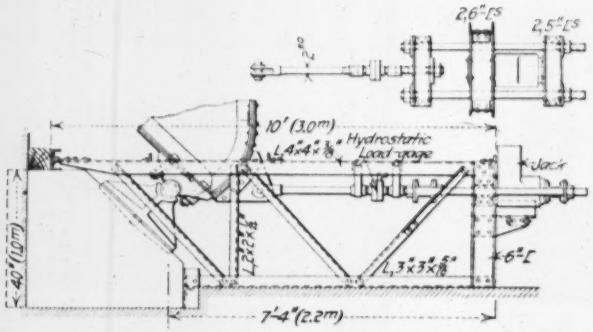



FIG. 2. HORIZONTAL JACKING DEVICE

falsework above true position and then was lowered upon a rocker cradle at one end, the other end being seated on its permanent shoe. At the same time a horizontal jacking frame was placed in position against the abutment, and its pulling rods were connected to hitches near the base of the arch leg. Force was first applied through a hydraulic jack, but later a screw jack was substituted and proved more convenient. The pull was measured by a 20-ton load gage of the water-displacement type, inserted in the pulling rod. The thrusts actually put into the several arches, determined in advance by computations from the known weights, etc., ranged from 10 to 13 tons. When the desired pull was applied by the jack, the shoe was put in place against the pin of the arch and blocked and grouted in position, thus holding the arch in the precise position required to give the correct thrust.

## Steam-Hammer Pile Formulas; Suggestions and Queries

By A. M. Knowles*

I read with much interest the article entitled "Test Comparing Steam and Drop Hammer Pile Formulas," *Engineering News*, Jan. 6, 1916. This article is of particular interest, as there has been so little material available concerning the actual relative performance of ordinary drop hammers and gravity-drop steam hammers, the latter being (I assume from its description) the type of steam hammer used at the Pearl Harbor and Mare Island dry docks. There seems to be still less reliable information available concerning the performance of the double-acting steam hammer, which has come into quite general use, as compared with either of the two other types of hammer.

While the theoretical part of the *Engineering News* formulas is generally accepted, the results of the formulas depend to a great extent on the constant used in the denominator, which can only be determined with any degree of accuracy by actual comparisons of tests such as Mr. Gayler has made. It does not seem possible that this very important field of engineering has really been neglected to the extent that would appear from any literature there is available. Undoubtedly there are engineers who have conducted tests along these lines with the view of developing formulas that will give reasonably close results when applied to piles driven under the same general conditions with each of the three types of hammers referred to. The writer, at least, and probably others would be pleased to hear of such experience through the columns of *Engineering News*.

The article referred to states: "At Pearl Harbor the average penetrations per blow by steam hammer and by drop hammer at the same attained depth were almost exactly proportional to the respective energies of the blow." From this comparison would it not be correct to assume that the constant in the denominator of the generally accepted gravity-drop steam-hammer formula depends almost entirely upon the interval between blows? Likewise in the original *Engineering News* formula for the ordinary drop hammer the constant 1 was, I understand, assumed arbitrarily as a compensation for the extra resistance caused by the settling of earth around the pile between blows, over other ordinary resistances.

Assuming that the steam hammer used by Mr. Gayler struck 60 blows per min. and using his constant 0.3, the ordinary drop hammer would be striking 18 blows per min., and it would be interesting to know if this relation did obtain.

The formula for the gravity hammer

$$P = \frac{2Wh}{s+1}$$

may be written in the following form to make it applicable to the double-acting steam hammer:

$$P = \frac{2h(Am + W - b)}{s+k}$$

in which

$P$ = Safe bearing load, pounds;
$h$ = Effective length of stroke, feet;
$A$ = Effective area of piston, square inches;

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$m$ = Mean effective pressure, pounds per square inch;
$W$ = Weight of moving part, pounds;
$b$ = Total back pressure;
$s$ = Penetration under last blow, inches;
$k$ = Constant.

The uncertain values in this formula are $m$, $b$ and $k$.

A manufacturer's catalog in commenting on the advantage of a double-acting steam hammer over a gravity steam hammer states: "The free fall of the hammer being accelerated by the steam pressure, the number of blows per minute is increased while the force of the blow is maintained." Therefore, with the force of the blow maintained, the constant should be decreased in proportion to the number of blows struck per minute, and assuming 60 blows per min. for a gravity-drop steam hammer and 100 blows per min. for a double-acting steam hammer, both hammers doing the same work per blow, and using the constant 0.3 as determined in the *Engineering News* article for the gravity-drop steam hammer, would not the constant for the double-acting steam hammer be in the proportion of $\frac{60 \times 0.3}{100}$, or 0.18? This would naturally give much higher bearing value for equal penetration per blow.

For the double-acting steam hammer the total force upon the ram consists of its weight and the steam pressure on the cylinder, which is usually assumed as the mean effective pressure for the entire length of the stroke with a suitable boiler pressure. This mean effective pressure varies naturally as the throttle lever is changed to regulate the speed of the hammer, to say nothing of the variation of the boiler pressure; and so, would not the constant depend upon the man at the throttle lever? Also, from actual experiments with indicator diagrams it has been found that the mean effective pressures so assumed are erroneous and much higher than found in actual practice. It would then seem that a suitable formula that could be used with the double-acting steam hammer is still more variable and complex than for the other two types of hammers and should certainly be given due consideration.

## Some Better Kutter's Formula Coefficients

By Robert E. Horton*

The ordinary category of coefficients of roughness for use in the Kutter formula (which may be found in various books on hydraulic calculations) lists, as a rule, not more than 12 to 25 classes of pipes, conduits and channels, with one coefficient of roughness for each. This number of classes of channels is too limited; and the number of values of the coefficient and the range of value are not sufficient to cover modern requirements in calculation. Furthermore, various types of channels and conduits now frequently used are not listed at all in the older schedules.

It is well known that to select the proper coefficient of roughness to be applied under given conditions in connection with the Kutter formula requires considerable experience and judgment. The working engineer is loath to spend his time hunting through original sources of data and experiments, and the range of intermediate values between those given in the existing schedules is so wide as to leave large room for error or diversity of opinion.

Many values of the coefficient of roughness for new conditions have been recently worked out and published. These newer values relate to flow in concrete pipes and conduits; irrigation ditches, canals and flumes; circular, semicircular and other steel pipes and channels; and natural river channels.

In order to facilitate the rapid selection of satisfactory values of the coefficient of roughness and to standardize office-engineering practice in such calculations, the writer has made out the accompanying table of coefficients. In this table four coefficients of roughness are given for each class of channel or conduit, the intention being to cover the usual range of values occurring in actual experience and also to include the values that may be expected to apply (1) to new work, in the best order and condition, and (2) to pipes or channels that have been in use for a greater or less length of time.

Obviously it is necessary to use different values of the coefficient of roughness for the same channel, according to the conditions to be met. For example, if the thing to be determined is the actual quantity of water that a pipe will carry when new, then the coefficient for the pipe in perfect order should be used. However, if as is often the case in designing, it is desired to provide pipe or conduit capacity sufficient to convey a certain amount of water when the pipe or conduit has been in use for a term of years, a higher coefficient of roughness will in general be required.

Existing schedules of coefficients of roughness do not, as a rule, contain any values exceeding about 0.040, which is the highest value commonly given for natural river

NEW TABLE OF n FOR KUTTER'S FORMULA OR R. E. Horton

| Surface | Perfect | Good | Fair | Bad |
|---|---|---|---|---|
| Uncoated c.-i. pipe | 0.012 | 0.013 | 0.014 | 0.015 |
| Coated c.-i. pipe | 0.011 | 0.012* | 0.013* | |
| Commercial w.-i. pipe, black | 0.012 | 0.013 | 0.014 | 0.015 |
| Commercial w.-i. pipe, galv. | 0.013 | 0.014 | 0.015 | 0.017 |
| Smooth brass and glass pipe | 0.009 | 0.010 | 0.011 | 0.013 |
| Smooth lockbar and welded "OD" pipe | 0.010 | 0.011* | 0.013* | |
| Riveted and spiral steel pipe | 0.013 | 0.015* | 0.017* | |
| Vitrified sewer pipe | 0.010<br>0.011 | 0.013* | 0.015 | 0.017 |
| Glazed brickwork | 0.011 | 0.012 | 0.013* | 0.015 |
| Brick in cement mortar; brick sewers | 0.012 | 0.013 | 0.015* | 0.017 |
| Neat cement surfaces | 0.010 | 0.011 | 0.012 | 0.013 |
| Cement mortar surfaces | 0.011 | 0.012 | 0.013* | 0.015 |
| Concrete pipe | 0.012 | 0.013 | 0.015 | 0.016 |
| Wood-stave pipe | 0.010 | 0.011 | 0.012 | 0.013 |
| Plank Flumes: | | | | |
| Planed | 0.010 | 0.012* | 0.013 | 0.014 |
| Unplaned | 0.011 | 0.013* | 0.014 | 0.015 |
| With battens | 0.012 | 0.015* | 0.016 | |
| Concrete-lined channels | 0.012 | 0.014* | 0.016* | 0.018 |
| Cement-rubble surface | 0.017 | 0.020 | 0.025 | 0.030 |
| Dry-rubble surface | 0.025 | 0.030 | 0.033 | 0.035 |
| Dressed-ashlar surface | 0.013 | 0.014 | 0.015 | 0.017 |
| Semicircular metal flumes, smooth | 0.011 | 0.012 | 0.013 | 0.015 |
| Semicircular metal flumes, corrugated | 0.0225 | 0.025 | 0.0275 | 0.030 |
| Canals and Ditches: | | | | |
| Earth, straight and uniform | 0.017 | 0.020 | 0.0225* | 0.025 |
| Rock cuts, smooth and uniform | 0.025 | 0.030 | 0.033* | 0.035 |
| Rock cuts, jagged and irregular | 0.035 | 0.040 | 0.045 | |
| Winding sluggish canals | 0.0225 | 0.025* | 0.0275 | 0.030 |
| Dredged earth channels | 0.025 | 0.0275* | 0.030 | 0.033 |
| Canals with rough stony beds, weeds on earth banks | 0.025 | 0.030 | 0.035* | 0.040 |
| Earth bottom, rubble sides | 0.028 | 0.030* | 0.033* | 0.035 |
| Natural Stream Channels: | | | | |
| (1) Clean, straight bank, full stage, no rifts or deep pools | 0.025 | 0.0275 | 0.030 | 0.033 |
| (2) Same as (1), but some weeds and stones | 0.030 | 0.033 | 0.035 | 0.040 |
| (3) Winding, some pools and shoals, clean | 0.035 | 0.040 | 0.045 | 050 |
| (4) Same as (3), lower stages, more ineffective slope and sections | 0.040 | 0.045 | 0.050 | 0.055 |
| (5) Same as (3), some weeds and stones | 0.033 | 0.035 | 0.040 | 0.045 |
| (6) Same as (4), stony sections | 0.045 | 0.050 | 0.055 | 0.060 |
| (7) Sluggish river reaches, rather weedy or with very deep pools | 0.050 | 0.060 | 0.070 | 0.080 |
| (8) Very weedy reaches | 0.075 | 0.100 | 0.125 | 0.150 |

* Values commonly used in designing.

*Consulting Hydraulic Engineer, 57 North Pine Ave., Albany, N. Y.

channels containing stones or weeds. In the experience of the writer, and that of other engineers, coefficients of roughness for natural river channels often run much higher than 0.040 and are not uniform for the same channel at different stages of the stream. A natural river channel that has, for example, a coefficient of roughness of 0.035 in a certain reach at bank-full stage may at lower stages have a coefficient of roughness as great as 0.060 or even higher. This is owing, in general, to the fact that in determining the coefficients of roughness in natural river channels the entire cross-section as determined by soundings is taken into account, whereas if the cross-section is irregular there may be pockets or holes in the bed of the stream containing slack water at lower stages, so that the full area of section is not effective. Similarly at low stages of a stream there is likely to be more or less fall that is not effective as slope and that does not contribute to producing velocity.

Coefficients for natural river channels given in the table have been derived largely from unpublished experiments of the writer on the Mohawk, Black and Hudson Rivers in the State of New York. They have been confirmed by similar work of other engineers elsewhere. The matter of variation of the coefficient of roughness is more fully discussed in an article by the writer in *Engineering News*, Apr. 29, 1915, p. 827.

Experience shows that the coefficient of roughness for excavated channels of the same slope and cross-section is dependent upon the method of excavation. Additional experiments are needed to determine the specific coefficients for channels in earth excavated by different methods of dredging, etc.

More detailed lists of coefficients of roughness derived from recent experiments and utilized in preparing the small table here given may be found in "Report of the Florida Everglades Engineering Commission," Bulletin No. 194, United States Department of Agriculture; "Flow of Water in Irrigation Channels," by Fred C. Scobey, Bulletin 194, Colorado Agricultural College Experiment Station; "Frictional Resistance of Waterways," by Cohn, Trimble and Jones; and an article by Samuel Fortier, in *Engineering and Contracting*, Jan. 27, 1915.

## Drilling Methods in Driving 6-Ft. Tunnel in Granite

By S. W. Symons*

One element of the new sewage-disposal system for Portchester, N. Y., is a tunnel 900 ft. long and 5x6 ft. in cross-section, in granite. In driving the tunnel, hand hammer drills were employed, mounted on the crossarms of a center column, as illustrated in Fig. 1. The drills were cleaned by water from the city mains. The advance averaged 200 ft. per month of 25 working days, the record month being 210.5 ft.

The contract for the tunnel was let to the Daly & Merritt Co., of Portchester. W. C. Brennan was Superintendent for the contractors and was responsible for the make-up of his organization and for the methods used in driving the tunnel. Borings were made along the line, and early in April, 1915, ground was broken with steam drills at the extreme southern point. The shaft was completed and the heading started on May 8, 1915, when the steam drills were discarded and mounted "Jackhamers" were substituted.

Air was furnished by a 12x12x16-in. Rand straight-line compressor taking steam from a 50-hp. portable Ames boiler. Steam pressure was 90 lb., the air pressure 80 lb. The air was led through a 1½-in. pipe a distance of 60 ft. to the shaft and thence to the heading. The capacity of the compressor was nearly double that required by the drills, with the result that it was run most of the time at only 88 r.p.m.—a little over half its rated speed.

At the time of the writer's visit the tunnel had progressed 530 ft. northward, at which point soft ground was encountered; and it was decided to move the entire plant to the north end of the contract and drive back, in order to ascertain the exact extent of the soft material.

Fig. 2 is a diagram of the round. No. 1 drill, on the upper arm, puts in holes 1 to 9 in the upper half of the

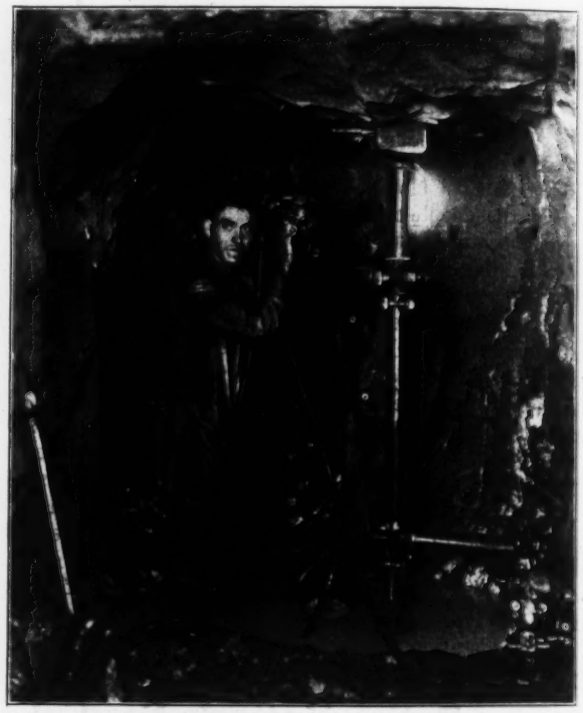


FIG. 1. HEADING OF PORTCHESTER SEWER TUNNEL, SHOWING MOUNTING OF DRILLS

heading, and No. 2, on the lower arm, drills holes 1-A to 8-A. No. 2 drill, as a rule, has one less hole to drill than No. 1, owing to a more awkward position. Two drillrunners, a nipper and two muckers constituted the tunnel crew for each shift.

Holes 1-A to 8-A in the lower half of the heading were loaded first and fired together; then holes 5, 8 and 9 in the upper half and, lastly, holes 1, 2, 3, 4, 6 and 7. All holes were bottomed at 1⅝ in., 1¼-in powder being used.

The average footage was 4 ft. per round, but when the tunnel broke high, allowing the use of a 6-ft. steel for holes 3-A, 4-A, 7-A and 8-A, as much as 5 ft. could be pulled. The total time for drilling the round, which included removing the heading muck far enough back to set up the drill column, amounted to from 4 to 5 hr., the loading and shooting consuming about 3 hr. Water, when encountered, was handled by a simple steam siphon. Drill steel was sharpened by hand. Crossbits were used.

*388 Henry St., Brooklyn, N. Y.

Track was laid to within a few feet of the heading, to facilitate mucking operations. The muck was loaded into a bucket, set on a trolley, pushed by hand to the shaft, the bucket hoisted to the top and the muck dumped close beside the shaft. A 2-drum hoist was used.

The powder used was gradually reduced to 10 lb. per yd. of solid material—approximately 10 lb. per ft. of

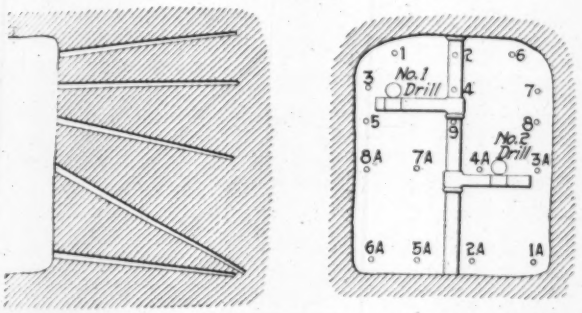



FIG. 2. LOCATION OF HOLES IN A DRILLING ROUND

advance. The rock, for the most part, broke rather small for economical handling, but this was to be expected, owing to the small size of the tunnel and the necessarily close spacing of the holes. The dynamite was 60%.

## City Bought Bridge Materials in Advance

Novel methods are being used in the City of Dayton in preparing for building three new concrete bridges—Fifth St. bridge across the Great Miami, and Webster St. and Keowee St. bridges over the Mad River. Bids are just being called for. The city manager last fall anticipated an advance in the price of steel bars and contracted for some 800 tons at a figure below $30 per ton base, f.o.b. Dayton. A contract was also placed for 53,000 bbl. of cement. Under these contracts both the cement and the reinforcing steel will be furnished by the city. Steel is now some $20 higher.

Bids are being asked by unit prices instead of by lump sum. The approximate quantities involved in the three bridges as estimated at present are given in the accompanying table. The designs were made under the general supervision of Service Director Barlow. Paul D. Fuqua, engineer in charge of bridges for the city, has direct charge of the design and construction. A local committee of architects coöperated in the architectural features. Harrington, Howard & Ash, Kansas City, were retained as consulting engineers.

APPROXIMATE QUANTITIES INVOLVED IN THREE DAYTON BRIDGE CONTRACTS

| Item | Webster | Keowee | Fifth |
|---|---|---|---|
| 1 Dry excavation, cu.yd. | 3,500 | 5,500 | 6,100 |
| 2 Wet excavation, cu.yd. | 2,100 | 2,000 | 6,100 |
| 3 Timber piles under masonry, lin.ft. | 4,700 | 7,500 | 9,500 |
| 4 Drainage system, lump sum | | | |
| 5 Porous fill over arches, cu.yd. | 3,500 | 3,000 | 3,700 |
| 6 Embankment for street approaches, cu.yd. | 6,000 | 12,000 | |
| 7 Cement, to be stored and handled, bbl. | 9,950 | 9,300 | 16,000 |
| 8 Reinforcement bars, to be handled, bent and placed, tons | 250 | 225 | 340 |
| 9 Class A concrete in spandrel and abutment walls and around ducts under sidewalk, cu.yd. | 525 | 500 | 600 |
| 10 Class A concrete in arch rings and in arch-ring anchorages in piers and abutments, cu.yd. | 2,800 | 2,300 | 3,600 |
| 11 Class B concrete in pier bases, cu.yd. | 1,010 | 1,050 | 3,100 |
| 12 Class B concrete in pier shafts and gravity-type walls, cu.yd. | 640 | 650 | 2,800 |
| 13 Class B concrete in pavement subbase, cu.yd. | 315 | 275 | 500 |
| 14 Class C concrete in abutments, cu.yd. | 2,050 | 2,250 | 2,500 |
| 15 Handrail above top of fascia, lin.ft. | 925 | 750 | 1,220 |
| 16 Cement curbs for roadway, 1:2:4, lin.ft. | 860 | 800 | 1,400 |
| 17 Cement sidewalks, 4-in., 1:2:4, sq.ft. | 7,700 | 6,100 | 11,000 |
| 18 Exposed concrete surfaces to be dressed by rubbing, sq.ft. | 11,500 | 10,000 | 10,000 |
| 19 Exposed concrete surfaces to be dressed by patent hammer or bush hammer, sq.ft. | 1,500 | 1,200 | 2,200 |
| 20 Arch rings and walls to be waterproofed, squares | 270 | 200 | 450 |
| 21 Cast-iron lamp standards to be set | 16 | 16 | 28 |
| 22 Trolley poles to be set | 8 | 4 | 12 |
| 23 Electrical ducts to be laid, single duct-ft. | 19,317 | 9,800 | 29,000 |

All cement and reinforcement bars for this work will be furnished by the city. Bids for constructing these bridges will be received on or about March 15, 1916.

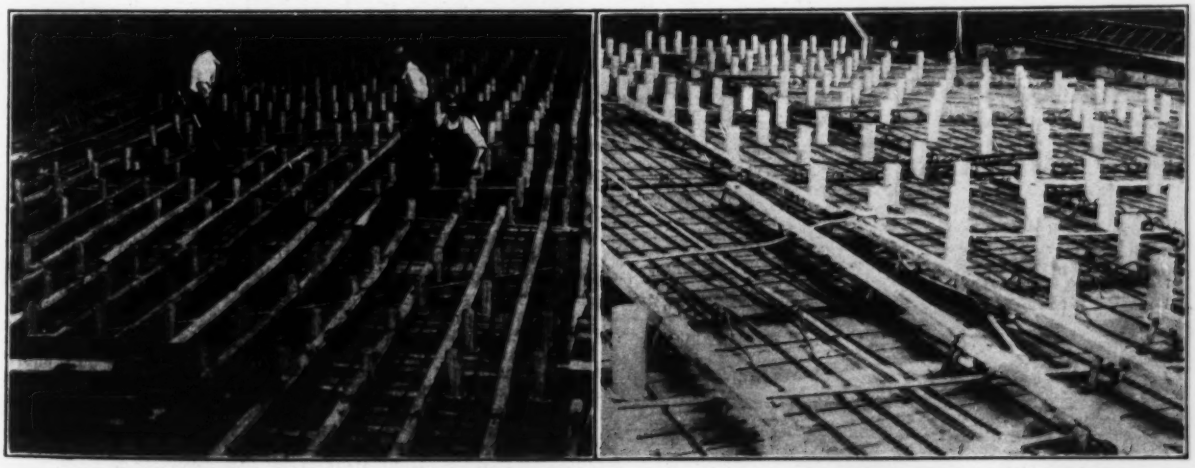
## Placing Warm-Air Registers in a Concrete Floor

By Arthur C. Hewitt*

The accompanying photographs show the construction work preliminary to pouring large reinforced-concrete floors in a Cincinnati theater that has warm-air registers under each seat on the first floor and on the balcony. On account of the great number of such registers some trouble in avoiding structural elements was anticipated.

In building the floors the first operation was to erect the forms and place the reinforcement. Then the center of each seat was located on the forms. There were only a few register tubes that had to be omitted because of interference with beams. After the seat centers were located, the contractor placed a galvanized-iron tube about 4 in. diameter by 10 in. long for each register. Next, the electric conduits were placed. Then the wooden sleepers

*Engineer, Security Cement and Lime Co., Hagerstown, Md.



TWO VIEWS SHOWING FORMS FOR WARM-AIR REGISTERS IN REINFORCED-CONCRETE FLOOR

were set in place, supported on iron brackets made of bar iron about 3/8x1 in., bent into shape as shown in the photographs. The iron supports were left in the concrete, which was delivered in buggies, the runways for the buggies being placed on the wooden sleepers. However, before the concrete was poured, each galvanized-iron tube was carefully filled full of sand, so that no concrete would get into it. After the concrete had set and the forms were removed, there was a 4-in. hole in the floor for almost every seat. Over these holes were placed registers.

A wooden floor was built upon the sleepers set in the concrete, so that the seats could be attached to it easily. The heating plant was below the main floor and consisted, in the main, of a boiler, steam coils, an air washer and humidifier and a large electrically driven blower. The heated air was blown into a plenum chamber under the main floor, from which it was conveyed directly to the little cast-iron registers under the seats. About 25 cu.ft. per min. of heated free air is blown into the theater for each person; and as it enters from so many points the audience is not annoyed by a concentrated blast as is the case with large registers.

## Safety Scaffolds for Buildings

For the protection of painters and others working on swinging scaffolds the Illinois Factory Inspection Department and the Chicago Building Department have issued rules as to the construction of such scaffolds. Investigation showed that numerous accidents resulted from the breaking of the 2x4-in. wood bolsters of the platform or from the stirrups slipping off the bolsters. The use of wire rope is made compulsory. The essential part of the rules is as follows:

No rope of any kind is allowed for tuck pointers' scaffolds, but they must have rigid bails of ¾-in. round or square steel, forged or welded in one piece, as shown in the upper view of Fig. 1. A metal clevis fastened around the bail and one of the crosspieces of the scaffold prevents the bail from slipping over the end of the scaffold. If braces are used, they must not be nailed or screwed, but must be fastened by a hook bolt, no part of the scaffold being cut or bored in order to attach them.

The bails and stirrups of painters' scaffolds may be either of wire cable with wood putlogs, as shown in the lower view of Fig. 1, or of steel bars, as shown in the upper view. All bails, stirrups and slings must be of ⅝-in. galvanized-steel cable composed of 6 strands of 12 wires each. This large size is specified in order to allow for corrosion and careless use. Loops and stirrups for the use of bolsters must be spliced. Beneath the platform must be an additional safety cable spliced to the stirrup or bail. This will hold the platform if the bolster should break. Putlogs and bolsters must be of hickory or white oak, not less than 2 in. wide and 4 in. deep. At not less than 3 in. from each end and at the middle of the depth must be a ¾-in. bolt 6 in. long, with an iron washer on each end. Between each washer and the bolt head or nut must be a pipe sleeve 2 in. long, forming projections which prevent the bail or sling from slipping off the end of the bolster. The end of the bolt must be battered or riveted so that the nut cannot be removed. The construction of the safety bail of steel rope is shown in Fig. 2.

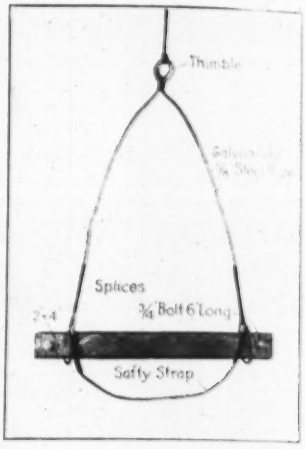



FIG. 2. SAFETY BAIL FOR SWINGING SCAFFOLDS

Some large buildings in Chicago are equipped with permanent scaffold runways on which are trolley hoists for carrying swinging platforms used by window washers, painters, tuck pointers, etc. The office building of Swift & Co. has an I-beam runway below the cornice and about 14 in. from the wall, supported on brackets. This is safer and more economical than placing and shifting roof timbers or hooks from which to suspend the scaffolds. It also prevents the nuisance due to the periodical opening of windows by the cleaners.

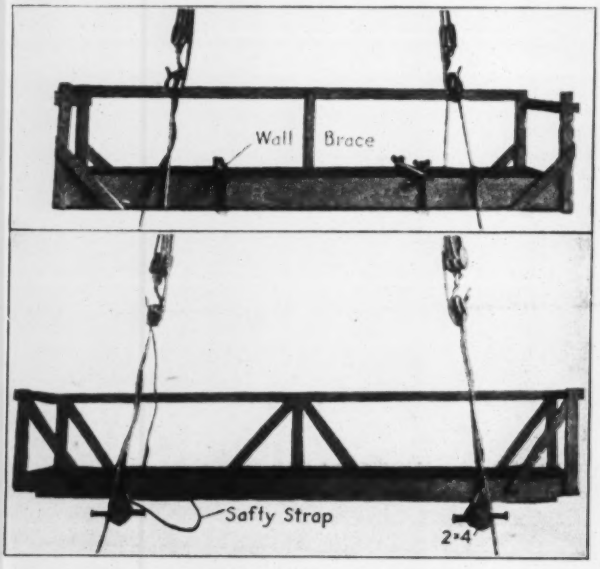



FIG. 1. SAFETY SWINGING SCAFFOLDS

## NOTES

**Teaching Lettering by Correspondence**—Something of a novelty is the announcement of a correspondence course in lettering. C. W. Reinhardt, of Arlington, N. J., for 20 years in charge of the drafting department of "Engineering News," is the author of this innovation. While it might be thought that teaching lettering, like teaching skating, is not amenable to mail-bag methods, the instruction sheets carry the spirit of the "Reinhardt system" quite remarkably.

**Street Cleaning at Utica, N. Y.,** is contracted for on the basis of great squares (10,000 sq.ft.) per day, and is done regularly during the summer and when directed in winter—about 225 days per year. The city is divided into two districts, in one of which there are daily cleaning and an hourly patrol. Here the contract price is 41c. per great square per day. In the second district there are a cleaning every two days and a patrol every 2 hr. Here the price is 26c.

**Blueprint Paper May Get Expensive**—The price of blueprint paper and blueprints is likely to advance materially in the near future. This is due to the increase in the price of all chemicals employed in the blue process and also to the big advance in paper. Red prussiate of potash has been made exclusively in Germany. Before the war it sold for 19c. per lb.; today it is practically unobtainable at $7 per lb., or an advance of 3,500%. Citric acid, from which the citrates of iron and ammonium are made, has advanced 150%; and oxalic acid, from which the oxalates of iron are made, has advanced 1,000% in the same time. The rise in the price of the cheaper grades of paper in which wood pulp is used is due to the pulp shortage caused by the scarcity of all paper-making chemicals and to the Swedish embargo prohibiting the exportation of chemical wood pulp. The better grades of paper, made of all-rag stock, have likewise advanced sharply, due also to the price of paper-making chemicals and to the shortage of supplies of rags.—New York "Commercial."

# Editorials

## Building Reinforced Concrete on the Unit System

From one point of view there is nothing new about the so-called unit system of concrete construction. It consists in general, as is well known, of casting, at a central casting yard, the individual pieces of the structure to be erected, transporting them to the place where they are to be used, hoisting them into position with some type of crane and making the final structure monolithic by overlapping the reinforcement of the units and grouting the open space between them.

It is of interest to record, however, that this system of construction appears to be coming into increasing favor among engineers. Elsewhere in this issue is described a 3-mi. trestle on the California state highway near Sacramento, just completed, in which the unit system has been applied to great advantage.

Another noteworthy use of the unit system is in the erection of railway train-sheds of the umbrella type. In the last year and a half the Southern Pacific Co. has erected sheds of this type at its station in Los Angeles and also at its station in San Francisco. The total extent of the umbrella train-shed roofs at these two terminals is about 8,000 lin.ft. At Denver the Denver Union Terminal Railway Co. has just completed about 7,000 lin.ft. of umbrella platform roof, erected on the unit system, at the Denver Union Station.

Inspection of the workmanship in these new train-sheds and in the Sacramento trestle shows results very favorable to this method of construction. Better work is secured, on the average, where concrete is made and mixed at a central yard than where it is mixed and poured on the job. The unit construction, of course, facilitates the use of the same forms over and over again, thereby greatly reducing the cost of form work. It greatly facilitates the bending and placing of reinforcing steel, since the steel required is the same for each unit and the workmen rapidly grow expert in forming and securing it properly in the molds. The thorough seasoning of the units before they are hoisted and placed in the structure helps to insure against the accidents that occasionally occur when concrete is poured in place. By a proper arrangement of the reinforcing steel it is possible to handle the units and hoist them to place in the structure more cheaply probably than the concrete materials themselves could be hoisted and poured.

In the structure referred to, moreover, it has been found possible so to arrange the reinforcement that the final structure, although built up of assembled units, is truly monolithic. It will be recognized, of course, that such structures as trestles and train-shed roofs lend themselves especially well to the unit system of construction, whereas in ordinary building construction the final monolithic result is not so easy to obtain.

It will be admitted also that where the particular appearance of the structure is a matter of importance there may often be difficulty in building by the unit system. The Sacramento trestle, for instance, could hardly be praised for its beauty, so far at least as a side view of the structure is concerned. Appearance was doubtless considered unimportant here, for the trestle is in the midst of a vast marsh, so that no one can see the trestle except travelers on the railway near-by.

Had it been necessary to consider architectural appearance in the Sacramento trestle, it would probably have been necessary to resort to pedestal piers and longer spans, which would have precluded the use of the unit system.

On the other hand, it may be pointed out that an engineering firm on the Pacific Coast has worked out a design of unit construction for three-hinged reinforced-concrete arches by which, as has been described in this journal, the half ribs are cast separately and hoisted into place by a derrick. In the design of ornamental bridge railings also it is very common to design a balustrade made up of individual units which can readily be cast from a mold that may be used over and over again; in this way a design of considerable architectural pretensions can be made at comparatively moderate cost.

## Concrete Roads Carpeted with Bituminous Material

A few years ago a sensation was created in street-paving circles by the announcement that a new type of permanent paving for streets and roads, which could be laid at a cost of about a dollar per square yard, had been developed in the City of Ann Arbor, Mich. The new pavement consisted of concrete covered with a thin carpet of bituminous material. Immediately the new paving was tried in many different places and with results that were almost uniformly unsatisfactory. The trouble generally experienced was the tendency of the bituminous carpet to peel off the concrete base. In Ann Arbor itself there was much trouble, but it appears from the first-hand account of Ann Arbor's experience given elsewhere in this issue that the use of this kind of pavement has by no means been given up by that city. The troubles experienced at Ann Arbor are claimed to have been due more to faulty construction than to inherent defects in this type of pavement, although it is frankly admitted that the bituminous top has required frequent repair.

This recital of Ann Arbor's experience is of especial interest, because it comes at the same time as the meeting at Chicago of the second National Concrete Road Conference, devoted to the discussion of methods of building concrete roads. If the engineers and cement manufacturers who have been active in the development of concrete roads believed that the thin carpet top for concrete roads had merit, they would long before this have undertaken its promotion, for the amount of cement required is nearly the same for this type of road as for the all-concrete type. Notwithstanding such negative

testimony there is reason to believe that the last word has not been said concerning the carpet top for concrete paving. Fifty men may fail in a given endeavor before one man supplies the single element needed for success.

The State Highway Department of California, under its chief engineer, Austin Fletcher, has scored a remarkable success in the use of a carpeted pavement, nearly a thousand miles of it having been laid. It has not, of course, demonstrated that this type of road could be made an equal success under the very different climatic and other conditions prevailing in the East. It has, however, shown that this type of road ought not to be discarded as a failure until it has been thoroughly tried in other parts of the country besides California.

The objections at once raised by advocates of the ordinary concrete road are: Why is not the concrete road without the bituminous top good enough? What is the advantage of going to the extra expense of the carpet coat on top of the concrete? It may be well therefore to state these advantages as demonstrated by the experience in California:

First, a bituminous-carpeted road is smoother and more elastic than one with a concrete top. In building a simple concrete road it is necessary to use a very rich mixture (1:1½:3 is standard practice), in order to secure a surface that will stand abrasive wear; but for roads with a bituminous top a leaner mixture may be used (1:2:4 is usual on the California state roads). The expansion joints, generally considered a necessity on concrete roads, are entirely omitted in the California roads that are to be carpeted. This makes quite a saving in cost. A further saving is made in the cost of finishing the road. No such pains and expense are necessary in finishing the road that is to have a traffic mat as on the road where the concrete itself must take the wear. Experience thus far indicates that the bituminous top is worn materially less by traffic than is a concrete surface.

It will certainly be agreed that all these things are important. Highway engineers have a constituency very difficult to please, and what is highly satisfactory in one section may awaken a storm of criticism in another. To a community that has long been used to ploughing through roads deep with the sticky mud of the Mississippi Valley, a new concrete road on which the city dweller and the farmer can run their motor cars at all seasons of the year seems like perfection itself. But build a concrete road in some sections of the East where automobile owners have become accustomed to smooth bituminous-macadam roads with an elastic rubber-like surface, and complaint is at once heard that the concrete road is too rough, that it is too hard and unyielding, that it wears the tires, that its glare is trying to the eyes.

From experience in riding over the California state roads it can be said that they have as perfect a surface as any automobilist could desire. They are so smooth, in fact, that they are habitually run over at railway speeds by motor cars. The number of cars per day is small on a large part of the mileage, but the high speed of the cars makes the service severe enough to test the quality of the surface; and very few bituminous-macadam roads would stand up as well under this fast traffic as do the California roads.

When we add to this the fact that the carpeted roads of California have been laid at an average cost of about $1.15 per sq.yd., or little if any more than the average cost of concrete roads in the East, the great advantage of this type, for California at least, appears to be demonstrated. Where frost in the ground and consequent heaving of road foundations are unknown, it is practicable to lay a concrete road only 4 in. thick; but it would be a risky experiment in New England. Even in California thicker concrete is used where adobe clay is encountered in the subgrade. The cheapness and excellence of California asphalt are incentives to the use of the bituminous top, while the high price of cement there makes the saving due to the use of a leaner mixture for the foundation concrete very important. Economizing in the thickness of the concrete is dictated not only by the cost of the cement, but by the fact that the stone has to be brought by rail in most localities.

All these, however, are minor matters compared with the one matter of making the bituminous carpet stick to the foundation concrete. Mr. Fletcher has succeeded in this on his California roads, where pretty much everyone else has failed. If this one difficulty can be overcome, there is almost certainly a big field for the carpeted concrete pavement in other parts of the country. Some claim the reason for the California success is the climate. Conditions are ideal in that climate for handling and placing bituminous mixtures, and the moderate range of temperature enables one to use an asphaltum that will not soften at the highest temperature and yet will not be brittle in the coldest weather.

Another reason for success is the practice of turning traffic over the concrete road for several weeks before the carpet coat is applied. The traffic grinds any adhering laitance or clay off the concrete, and the road is in condition to be swept perfectly clean to receive the hot asphalt. This practice has never been followed elsewhere, so far as we are aware.

It is worth mentioning, too, that the bituminous coat is applied by the state's own forces, although the concrete roads are built by contract; also, that the asphalt oil is applied hot—at a temperature of 250° to 350°—from a large tank wagon. The first coat of oil is applied at the rate of ¼ gal. to the square yard, giving a coat about $\frac{1}{4}$ in. thick. Upon this oiled surface are thrown fine crushed-stone screenings sufficient to absorb all the oil; the surplus is cleaned off, and the surface is rolled. Then a second coat of oil—¼ gal. per sq.yd.—is applied, and screenings carrying a small percentage of dust are applied. The finished coat is ⅜ to ½ in. thick and looks like an asphalt pavement.

There is reason to believe that the use of hard crushed stone instead of sand or fine gravel is one reason why the material does not roll or creep under traffic and separate from the concrete. It will be noticed also that the California carpet coat is considerably thicker than some of the carpet coats that have been unsuccessfully tried in various parts of the East.

It is fair to say that there have been a few cases where the California roads have given some trouble by peeling; also, that where the traffic is very dense, with many heavy trucks, some of the California roads have been built with the top coat 1½ in. thick. Lest it should be thought that an arid California climate is an essential to success, it may be said that a recent development is the sprinkling of the concrete before the hot oil is applied. It has been found that the oil sticks better if the con-

crete is slightly damp than it does if the concrete has even a slight coating of dust.

Mr. Fletcher states that his bituminous-carpet coat is applied to the California concrete roads at a cost of less than 1c. per sq.ft. and can be renewed, when partly worn, for less than ½c. per sq.ft. Costs would be higher than this, of course, in nearly all parts of the East; but if any such degree of success can be secured, a considerably higher cost can well be afforded.

It is well worth while for Eastern road engineers to try again on the carpet coat for concrete roads. Some of them, indeed, are already doing it. In New Jersey, for example, Col. E. A. Stevens, the State Road Commissioner, is experimenting with a wash of sulphate of zinc applied to the concrete before putting on the hot oil, and with reported excellent results in the way of firm adherence of the bituminous coat. Better construction and maintenance at Ann Arbor, more experience in California and the experimental and practical tests in other parts of the country may be expected within a few years to give data for more definite conclusions regarding bituminous-carpeted concrete roads than are yet warranted. Today it can at least be said that there is reason for being optimistic rather than pessimistic on the subject.

## City Manager's Experience with the Government of Abilene

For 18 months Abilene, Kan., enjoyed the distinction of being classed as a city administered by a city manager. The former city engineer, Kenyon Riddle, a young man from the University of Kansas, held the office of city manager under a board of three commissioners, including the mayor. In those 18 months the business of the little city was organized to run so smoothly that the commissioners decided there was no business left to require the services of a manager! So they have placed the former city manager on a *per diem* basis to do the city's engineering work.

The laws of Kansas do not specifically authorize a city-manager form of government, although the commission plan is provided for. The trouble at Abilene, therefore, was that the city manager was the creation of the commissioners; and these commissioners were salaried officials, each charged by law with the supervision of some city department. The mayor, who is chairman of the board, receives the munificent salary of $25 per month, and the other two commissioners $22 per month.

The ordinance creating the office of city manager was never legally adopted, but merely tacitly agreed to; so when the city manager commenced to shine in the administration of the city's affairs, the commissioners grew jealous of their prerogatives. The city was not big enough nor the business complex enough to require the services of all, so the city manager was the one to go.

The city manager found that the local electric company would operate the water-works pumping by electricity for one-third less than the city could operate it, so he leased it to the electric company. The city is so peaceful and righteous that the police force consists only of one day marshal and one night marshal; and the fire department consists of one piece of horse-drawn apparatus, so that the duties of the chiefs of the police and fire departments were not very onerous. There were no vice or social problems to work on, so it left little but the city engineering work to be done; and for the sake of economy, real or imaginary, the commissioners decided that this could be done cheaper on a *per diem* basis than at $1,200 a year, which was the city manager's salary.

There is a moral in this experience of Abilene, as well as an illustration of the old adage that too many cooks spoil the broth. While the experience proved that the city-manager plan is ideal for a small city like Abilene, it also proves that to be successful the office must have some foundation in law. Such powers as the city manager had were grudgingly bestowed by the commissioners, given one day and taken away the next, and whatever the city manager accomplished was only at the expense of much argument and persuasion.

## A Moribund Society Reviving

Those who went last week to Chicago to attend the obsequies of the American Concrete Institute were agreeably surprised. The victim was not dead, nor even dying. While hardly as lusty or as enthusiastic as it was in its youth, it still showed signs of a prospective rejuvenation, particularly if care is taken in the matter of diet and behavior.

To abandon metaphor, the institute is in a hopeful condition. Those in authority seem to realize that the pomp and ceremony of a major society are not part of their program and that the useful and quite necessary functions of the society can be taken care of in a much more simple manner than in the past. Freed, as it now is, from the necessarily hampering influence of solicited financial contributions, the society can proceed with such funds as its dues bring in to develop the standards of concrete construction with a single eye to the needs of the members. As these members increase, the service to them can be proportionately increased, until in time it can successfully realize the ambitious program which of late it has attempted with conspicuous failure.

In view of the necessary curtailment in effort for the coming year two definite recommendations might be made to the institute. The first is to leave out of the annual meeting all papers of mathematical complication. Such papers are distinctly not amenable to open presentation. At the same time search should be made for controversial subjects and effort made to promote discussion upon them. The second suggestion is that it is not the duty of the American Concrete Institute to prepare specifications or recommendations on subjects that are already under discussion by societies of wider scope, particularly when the institute's committee has to await the results of the larger society before taking final action. If economy of effort is necessary, here would seem to be a good place to start.

## Making Military Engineers

We noted last week the great enthusiasm with which some twenty-five hundred civil, mechanical, electrical and mining engineers in New York City started in a course of lectures aimed to help the transition of engineers from civil life into military—in the event of their need in a scheme of national defense. We note in our news columns this week the proposition of the governor of New York to build up a reserve corps of engineers.

Without in the least reflecting on the patriotism of all the engineers who are thus showing their desire to

prepare for military service we think we should caution them, and all others similarly inclined elsewhere, about the magnitude and discomfort of their task. Surely, it is well to turn aside all the easily discouraged and unfit early in the development of this movement, for the engineer troops are so vitally important that they must not be allowed to become a weak arm of the main force. We should say this even at the risk of seeming cold to any reasonable plan for the organization of our national defense—an attitude which we must immediately disclaim.

Captain Robin, in the opening lecture in New York, repeatedly harked back to the necessity of stern military training for the would-be engineer soldiers. We would fail in our patriotic duty did we not warn all office men—and the engineers behind this movement are largely tied down to their desks—that they are assuming a tedious, grueling task to get this military preparation. It means long hours of drill with arms, practice marches, camp hardships, short rations and rough fare, exposure to heat and storm. It means a program that only the physically fit should undertake.

There is no flag-waving about this sort of work—none of the glamour of parade to encourage drooping spirits. With a wise seclection of recruits the hard program will be carried through manfully, and the men will all show the true patriotism that we are sure they possess.

## When Assumptions Go Wrong

The foundation-girder problem discussed by two correspondents in this issue is a fair sample of the paradoxes that every little while rise up and bother the practicing engineer. Careful reading of the discussion and comparison with the problem as originally stated are well worth while. At the same time it is important to bear in mind that the question involved is by no means mere academic theorizing. In the particular case cited each foundation would require a ton of steel either more or less, depending on which view of the paradox prevailed. Dollars and safety are likely to be at issue in all these vexatious puzzles.

In dealing with such difficulties it will be found helpful to remember that most of them arise from a failure in the underlying assumptions—or approximations—of the reasoning employed. All engineering calculations are based on assumed conditions intended to picture the real (but variable and not definitely known) circumstances in normal cases. But in some particular application, whose appearance may not indicate that it is abnormal, the error of the standard assumptions may be very large, and the ordinary reasoning therefore may give misleading results. Then arise such paradoxical contrasts as in the foundation-girder problem.

# Letters to the Editor

## Making Military Engineers

Sir—Last week you heralded the course of military training which the engineers of New York City have enthusiastically plunged into. Now comes forward Governor Whitman with a proposal to organize a reserve corps of engineer troops out of the engineers in the employ of New York State. Undoubtedly, both these movements are highly patriotic; but I think we ought to make it very plain to everyone aching for military glory that it will be no royal road that he must travel.

My own view, from a considerable experience in state service and a pretty intimate acquaintance with a large proportion of the 1,200 engineers in the state employ, is that they undoubtedly possess the qualifications and education that would make it easier so to train them to be of value for military service along engineering lines than to train perhaps any other equal number of men that could be selected. Yet my feeling is that, until they are so trained, they would be of comparatively little value for field service.

The fact is that a large proportion of the engineers in state service have never done any work independently. They have always worked inside an office and know very little about practical mechanics or location operations. It would not be exaggerating matters to say that some of them could not saw a board squarely or drive a nail through a block without splitting it. What they need, and what they would have to have, to make them in any way efficient would be long hours and hard work under field conditions. They would be obliged to fell trees and, with absolutely unskilled labor, build temporary bridges in rapid order, dig trenches, put up entanglements and do similar work. Now, to take a group of men, of whom not one in two hundred perhaps has ever walked ten miles in a day over rough ground and who does not know the weight of a rod of barbed wire, and tell one of them to go to a certain place with a handful of men and put up a barbed-wire entanglement inside of 14 hr. would be perfectly useless.

I am not in any way deprecating engineers, but simply feel that they ought to come up to the public esteem for, and expectations of, them. I feel that they can and will do it, but they must not be expected to do it without some preliminary training that means a great deal more than simply reading books on military science, going out in khaki, living well in camp, wearing horn spectacles and looking wise.

VET.

Albany, N. Y., Feb. 18, 1916.

## Cutting a Pier in Two To Make It Stronger

Sir—The problem presented in *Engineering News* of Feb. 10, 1916, by George E. Strehan is one frequently met with in the design of footings for buildings. Unquestionably, the method shown in Fig. 2, with separate piers, would always be used if the column loads were the only loads to be considered. It frequently happens, however, in curtain-wall construction that the footings must be made continuous between the columns to carry the wall, as in the case of the corner column of a build-

in it, in which event the problem presented in Case 1 obtains. There was probably no serious contention on Mr. Strehan's part that "a part is stronger than the whole," which will hardly admit of argument in this instance.

The apparent reason for the query is the fallacious assumption that in Case 1 a uniform load is carried by

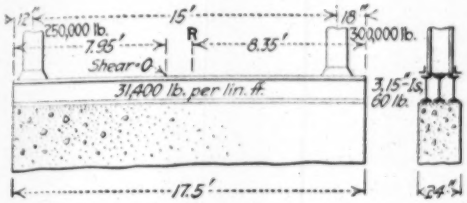



FIG. 1. FOUNDATION GIRDERS 130% OVERLOADED IF PRESSURE IS UNIFORMLY DISTRIBUTED

This and Fig. 2 reproduced from "Engineering News," Feb. 10

the beams throughout their length. This condition is impossible, on account of the deflection of the beams. In the example cited a uniform load of 31,400 lb. per ft. would cause a deflection at the center of the span of 0.21 in., figured on a simple span of 15 ft., provided the stresses were within the elastic limit of steel. It is somewhat less in the case given, on account of the overhang at the ends. The problem stated that the footings extended to rock, which permits no settlement at the ends, which would be necessary for the center to rise relatively and follow the curve that the beam would assume if uniformly loaded.

In Case 2 the load is not uniform on the piers, owing to the deflection of the beams, but it is customary to assume it so. The actual stress in the beams in this case is less than that produced by a uniform load on the pier, due to the fact that the center of pressure is not at the center of the pier. If the footings will safely carry the load, as in Case 2, why worry about getting on them a lighter load per square foot?

It is impossible to make any assumptions of elasticity of the concrete and rock foundations and construct a theoretical determination that will agree with the actual conditions. If the beams will really distribute the load in Case 1 over lengths equal to Case 2, which gives pressures within the prescribed working stresses for the concrete footing, it would seem reasonable to assume that the load acts only on this length, instead of designing them to distribute the load uniformly over their full length of bearing on the masonry, when the conditions of the problem make this an impossibility.

S. F. HOLTZMAN.

11 East 24th St., New York, Feb. 17, 1916.

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Sir—In the Feb. 10, 1916, issue of *Engineering News*, George E. Strehan discusses the case of a footing design in which two column loads, applied adjacent to the property line of a building lot, are carried by the ends of a steel distributing girder supported along its full length by a foundation wall. The intended purpose of the girder is to distribute the load over the rock foundation between the columns, but Mr. Strehan has shown that it is greatly overloaded, provided that it serves its purpose in uniformly distributing its load. However, if the center portion of the foundation wall is removed, the girder is of sufficient strength.

That the assumption of a uniformly distributed loading, with a consequent overstressed condition of the steel, is erroneous is shown by the following: A uniformly distributed load of 550,000 lb. applied to the beam arrangement, shown in Fig. 1, would cause a deflection of approximately 0.75 in.

Assume a foundation arrangement similar to that of Fig. 2, with the steel distributing girder removed and with a foundation pier 2 ft. square directly under the column load of 250,000 lb. The load on this pier would be 435 lb. per sq.in., and the total deformation or shortening due to this load, assuming a modulus of elasticity of 2,500,000 and the deformation to occur in a length of 60 in., would be approximately 0.01 in.

While the pier of comparatively small cross-section would be deformed only 0.01 in., the addition of the distributing girder and the solid foundation wall would apparently result in a further deformation of 0.74 in., the intensity of loading being at the same time reduced from 435 to 109 lb. per sq.in.

It may be concluded from these results that the distributing girder is not overstressed, that the foundation

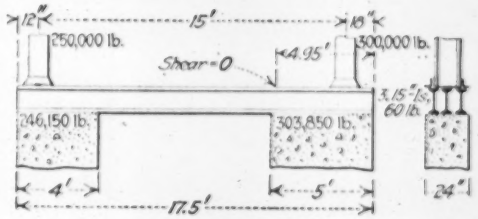


FIG. 2. SAME GIRDERS MADE SAFE BY CUTTING OUT MIDDLE OF PIER

as shown in Fig. 1 is not unsafe and that the arrangement of girder and continuous foundation wall does not result in a uniform distribution of load. In this particular case the girder receives only a small part of the load that it is capable of carrying.

G. M. WILLIAMS,
Bureau of Standards.

Washington, D. C., Feb. 15, 1916.

## Governmental Publication of Railway Statistics

Sir—On page 116 of *Engineering News*, Jan. 20, 1916, I notice some comment on the twenty-seventh annual report of statistics of railways. Your principal criticism is directed to the lateness of the date at which this report is issued. I quite agree with you that it is desirable to have information of this character published at as early a date as practicable, but the commission's publication of railway statistics is not confined to this report.

The commission receives monthly reports of railway earnings and expenses, which the companies are required to file with us within 30 days after the close of the month. We feel that we are getting the figures together and giving them to the press with all the promptness which can reasonably be expected, when we supply them within 40 days after the close of the month covered by the reports.

Under the statute the carriers have three months in which to file their annual reports of operation, and frequently it is necessary for them to secure additional time,

so that it is often the end of October before the reports of the larger carriers can be filed with the commission.

In order to give publicity promptly to these figures the division of statistics of the commission compiles and publishes each year what is known as the "Preliminary Abstract of Statistics of Common Carriers." This abstract is based upon the figures as reported, with such minor corrections as can be made without correspondence with the carriers, and we are usually able to send the copy for this compilation to the Government printing office early in December. Owing to the great pressure of work upon the Government printing office at this season of the year our work has to be subordinated to certain other printing, and it thus happens that this abstract cannot usually be published before some time in January, but we are generally able to get it into the hands of the public within seven months after the close of the year to which it relates, or within three months after most of the reports from which the figures are compiled have been filed with the commission.

Our annual volume, noticed in your issue of the 20th instant, is a much more elaborate report and necessarily takes a much longer period of time to prepare. If the figures as submitted by the carriers were in all instances correct, it could be published at a much earlier date, but unfortunately the reports of the carriers almost invariably contain discrepancies the detection of which requires a very careful examination in the office of the division of statistics and correspondence for the correction of errors and discrepancies thus discovered. This examination and the correction of the discrepancies require a considerable period of time, owing to the great number of reports that we have to handle. This largely causes the delay complained of. We have always thought it advisable, however, to put out in permanent form only such figures as have been carefully tested in order that they may be as accurate as practicable.

For the latest annual reports resort may be had to the preliminary abstract of statistics or to the reports themselves, which are public records, and for the current year's figures fairly satisfactory information regarding earnings and expenses may be derived from the monthly reports.

W. J. MEYERS,
Statistician.

Washington, D. C., Jan. 26, 1916.

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**The Increase in Automobile Registrations** in the State of Pennsylvania between 1914 and 1915 was nearly 44,000, or more cars than there were in the state in 1911. The receipts of the State Highway Department from the registration of 154,697 cars, 16,907 motor cycles, 2,297 tractors and trailers and from dealers' and chauffeur's licenses amounted to $1,631,169 during the current year. These numbers do not include the free registration of fire, police and ambulance motor vehicles.

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**Sewage Disposal for Los Angeles County, California,** is to be studied by a commission of sanitary engineers appointed by the County Board of Supervisors on Jan. 24. This action is the outcome of opposition to the proposed joint sewage-disposal works for Pasadena, South Pasadena and Alhambra, for which 600 acres was bought some weeks ago. The commission consists of Prof. Charles G. Hyde, University of California, who was named chairman; Thomas A. Gillespie, sanitary engineer of the State Board of Health; George P. Robinson, of the city engineer's office, Los Angeles; A. H. Koepig and A. M. Brosius. The commission has been instructed to study various methods of treatment in their relation to odors, flies and mosquitoes, and danger of the spread of disease germs; whether sewage effluent could be safely used for irrigation; the advisability of an ocean outfall and the depth and the distance from the shore at which treated sewage should be discharged.

# Rogers Pass Tunnel Method Chosen for Economy

BY J. G. SULLIVAN*

Under the heading "Two Tunnels to Make One," an editorial in *Engineering News* of Nov. 18, 1915, p. 993, discussed the method of driving used in the Rogers Pass tunnel of the Canadian Pacific Ry. The criticisms and questions of the editorial would appear logical and reasonable if speed were the only object in mind. But the most important factor—and the one that must govern all good engineering—is the factor of economy, and this alone was considered in making our decision. Time was a factor in this larger question of economy, but much smaller than some others. We considered time to be worth approximately $750 per day.

## THE EUROPEAN METHOD BARRED BY HIGH COST OF LABOR

In a circular letter sent out to contractors inviting them to bid, dated April 8, 1913, the following statements appeared:

> The necessity for this tunnel is so great and the expenditure so large that it would be worth considerable money to this company to have the tunnel completed as soon as possible. Therefore, everything else being equal, the party who will guarantee completion in the shortest time will be the party who will receive the work.
>
> I would be glad if you would give us prices on the European method of tunneling, which is to drive a very small heading and take out the bench, working from several headings into this small drift. Tunnels in Europe have been driven by this method at two or three times the speed that any tunnel was ever driven in the United States or Canada, and I would like to be able to place before the management figures for doing this work according to this method. I would be glad if you would state in your proposal the amount per day that you would be willing to have inserted in a contract to be paid as a bonus for time saved over the agreed time, the same amount to be exacted as a penalty for the time lost, being the time between the fixed day of completion and the actual date of completion. We are of the opinion that this sum should be about $750 per day.

The writer, however, was aware at this time that the chances of American contractors tendering a reasonable figure on the European method were very remote; in fact, after having studied the prices paid for labor in some of the large Swiss and Italian tunnels, we were forced to the conclusion that theirs was not a practicable method for this country, where labor is so expensive. On Mar. 13, 1913, the writer reported to the management his ideas on this subject. To quote from that report:

> Referring to the progress that we hope to make in the driving of the Rogers Pass tunnel. I advised you in my report of Oct. 22 regarding the relative speeds of driving tunnels on the American continent compared with those that have been driven through the Alps. I have given the matter considerable study since and have come to the conclusion that the European method of driving a small lower heading and stoping out the remainder of the tunnel is too expensive on this side on account of the difference in the cost of labor.

I then described in general the methods which we proposed and which we have practically followed out.

## ECONOMY REQUIRED A STEAM-SHOVEL METHOD

The method desired was one by which we would be able to handle the minimum amount of material by hand and the maximum amount by steam shovel. By driving a heading in the center of the excavation required for the completed tunnel, making this heading just large enough

*Chief Engineer, Canadian Pacific Ry., Winnipeg, Man.

so that we could handle steel that would reach from the perimeter of this advance heading to the perimeter of the completed tunnel, and then drilling holes at right angles to the axis of the tunnel we would be able to shoot at once the entire section required for the enlargement of the tunnel.

This shooting was done in the following manner: Six holes at the bottom of the tunnel having been previously shot out, six holes would be loaded in the next ring back in the bottom and all the side holes of the first ring up to a point a little over halfway up the sides of the tunnel. This method would continue until the tunnel began to fill up; and then some of the advance rings in the upper portion of the tunnel would be shot and this process continued until the tunnel was choked full of muck.

There were usually shot about six rounds (or 30 ft.) of holes before the steam shovel started to clean up. In some of the harder rocks that shot well it was possible to shoot a greater distance. The most shooting done at one time up to date is 84 ft. This method enabled us to handle about 85% of the total excavation of the tunnel proper with steam shovels.

This result was the prime object to be obtained, and how to accomplish it without interfering with the work of the shovel was solved by driving the "pioneer heading," or tunnel, to be used as a bypass for removing the muck from pioneer and advance headings, for ventilating ducts, air, water-pressure pipes, etc. The result was that work was carried on continuously at all points irrespective of shooting at other points, work only stopping at the local point at which shooting was being done. As there were no air pipes or water pipes or any obstruction whatever passing over the muck pile in front of the shovel, the shovel was able to do 85% of the total excavation almost as easily as if it had been an open cut.

### Excavation Done at Remarkably Low Cost

Our expectations have been more than realized, as is proved by the speed and the cost of the work. The cost of driving this tunnel through rock, including in this price the cost of driving 19,610 lin.ft. of pioneer tunnel, 12 crosscuts each about 40 ft. long, installation of plant (including freight), the proportionate cost of building about 5 mi. of temporary railway tracks, and other overhead charges, plus 10% on all expenditures, will amount to a little less than $5 per cu.yd. for excavation in the tunnel proper.

I may add further that, in reply to my invitation of April 8, 1913, the contractors who are doing the work, having in mind the method which was later adopted and which was suggested by myself and one of their engineers, bid $6.10 per cu.yd., with a time limit of 42 months from date of signing contract. Other responsible and supposedly expert tunnel contractors bid from $8 to $11.25 per cu.yd., with time limits varying from 42 to 48 months. I do not know what method these latter contractors proposed to employ, but I always presumed it would be some modification of the European method.

We also received an estimate of $5.50 per cu.yd. for tunnel excavation from a reputable American expert tunnel-contracting firm which would do the work on a percentage basis and would be willing to forfeit a large portion of its percentage if it did not keep the price within the estimate. However, it estimated the time at 58 months. At the speed this firm promised for driving tunnel in rock and figuring the time it took our contractors to get into rock at both ends on account of heavy cuts, it would have taken 8 or 9 months longer than the estimate, or something over 5½ yr. The method that it proposed to employ was to remove tunnel right from a face, loading muck by steam shovel.

### Shovels Advance More Than Three Miles per Year

Now as far as speed is concerned: On account of large cuts and soft ground at the approaches, the shovel doing the enlargement work did not reach the rock on the east end until Jan. 1, 1915, and on the west end until Feb. 1, 1915. On Dec. 19, 1915, when the heading was completed, the shovels were 2 mi. apart. Had the conditions been such that we could have got into the tunnel proper at earlier dates, I do not think there is any European record that would not have been broken. We have been driving tunnel at the rate of better than 3 mi. per yr. for the last year. During the month of January, 1916, the shovel in the east end made an advance of 946 lin.ft.

In conclusion I wish to say that in Europe, where drill runners, as I understand, get 90c. to $1.25 per day and laborers something like 75c., the method followed in driving the Loetschberg tunnel may be economical as well as rapid; but all the evidence we have goes to show that where we have to pay from $4 to $5.50 per day for drill runners and $2.25 to $3 for laborers the method we have adopted is by far the most economical.

## San Luis Rey River Floods of January, 1916

By Edward R. Bowen*

Marooned at Oceanside during recent floods in southern California, the writer embraced the rare opportunity thus afforded to study the January flood conditions of the San Luis Rey River. The results of the study are here given.

The San Luis Rey River flows in a general westerly direction for about 60 mi. and drains an area of 564 sq. mi. above its mouth at Oceanside. The river valley is quite level and has the same slope as the river. The confining rock at various points along the valley opens out and again closes in, thus forming a series of basins. The stream usually flows as a surface stream throughout its length, from a few days to a few months in the spring of each year.

The rainfall, while light along the seaboard, is considerable in the high mountain regions. San Diego, with a 65-yr. record, has a mean of 9.61 in. Oceanside has a 23-yr. mean of 11.15 in., while Nellie, at El. 5,350 ft., has an 11-yr. mean of 48.57 in. At the Warner dam site, El. 2,700, the mean rainfall for the past 4 yr. has been 30.34 in. The accompanying table shows the daily precipitation during the month of January at all the stations in and around the drainage area from which rainfall records are available at the present time. During the season 1915-16 the rainfall at Oceanside up to Jan. 1 was 4.06 in.—0.61 in. during November and 3.41 in. in the early part of December, so that the ground was in a moist condition at the beginning of January when the storms commenced.

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The recent floods occurred on Jan. 17 and 27. The precipitation up to Jan. 16 was generally a fine gentle rain, most of which was absorbed by the soil. The rains of Jan. 16 and 17 fell upon the water-soaked ground and appeared almost entirely as runoff. The flow of the river at its mouth on Jan. 15 was about 200 sec.-ft. No measurements were available for the next day. The river was in flood on Jan. 17, the crest reaching the mouth of the stream about noon of that date.

FIG. 1. GENERAL VIEW, SHOWING SAN LUIS REY VALLEY DURING FLOOD OF JAN. 27, 1916

### Gaging First Flood at Oceanside Bridge

An estimate of the flood was made by taking soundings across the river from the highway bridge at Oceanside. Current-meter measurements were made for a distance of 280 ft. out from the south bank, at which point the current was so swift that it was impossible to keep the meter in the stream. Floats were then timed both in the part of the stream over which current-meter measurements had been made and in the main current. The velocity in the main body of the stream was computed as 16 ft. per sec. and the runoff at 27,000 sec.-ft. The river at that time was 530 ft. wide, and its greatest depth was 14.5 ft.

Quantities of drift piled up against the Santa Fe trestle bridge, which crosses the river about 2,000 ft. west of the highway bridge at Oceanside, and finally washed out five bents. The San Luis Rey bridge, 4 mi. upstream, had about 100 ft. of its approach washed away. The Bonsall bridge, 17 mi. upstream, a concrete and timber structure, was washed out.

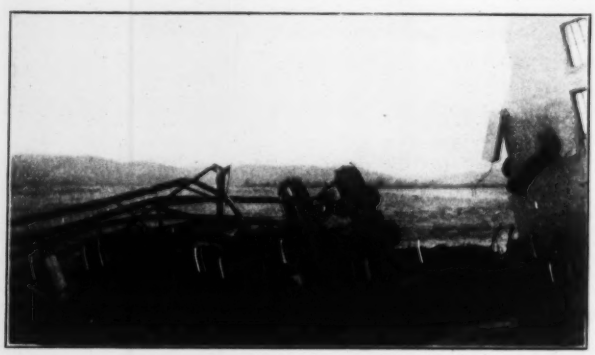

FIG. 2. WRECK OF PUMPING PLANT

The river spread over large areas of bottom and bench lands, destroying crops and in many places depositing large quantities of débris and silt. No lives were lost. All telegraph and telephone communication was cut off, and transportation by highway was interrupted for three days, while railway communication was cut off indefinitely. The roads up the river were passable only on horseback, and then with difficulty.

The river fell rapidly after 6 p.m., Jan. 17. On Jan. 18 an estimate was made in a manner similar to that previously described, except that the floats were timed and the station at which they were measured was noted. These surface velocities were then plotted as ordinates against the stations across the stream as abscissas, and intermediate velocities were taken from the curve. The velocity curve thus obtained was very consistent in its form. The result of this estimate showed 16,000 sec.-ft. in the morning and about 18,600 sec.-ft. in the afternon. Jan. 19 the river flowed 10,300 sec.-ft. in the morning and 10,200 in the afternoon. Further measurements were not available until Jan. 26, when the river flowed 1,757 sec.-ft. by current-meter measurements from the Oceanside bridge.

### Stream-Flow Measurements for Second Flood

On the afternoon of Jan. 26 the second division of the storm arrived at Oceanside. From this time up to 8 a.m., Jan. 28, the rainfall at Oceanside amounted to 3.62 in. The rainfall in the mountainous districts (see table) was substantially greater. This storm produced a flood of much greater volume than the preceding one.

A measurement of the river at the Oceanside bridge about 10 a.m., Jan. 28, showed 9,600 sec.-ft. An esti-

INCHES OF RAINFALL IN AND NEAR SAN LUIS REY RIVER DRAINAGE BASIN, JANUARY, 1916

| Date, Jan. | San Diego, El. 93 | Oceanside, El. 67 | Escondido, El. 657 | Warner's Dam, El. 2,700 |
|---|---|---|---|---|
| 1 | 0 | 0.26 | 0 | 0 |
| 2 | 0 | 0 | 0 | 0.23 |
| 3 | 0 | 0 | T* | 0 |
| 4 | 0 | 0.10 | 0 | 0 |
| 5 | 0.01 | 0.02 | 0.07 | 0.16 |
| 6 | 0 | 0 | 0.27 | 0.41 |
| 7 | 0 | 0 | 0 | 0 |
| 8 | 0 | 0.26 | 0 | 0.44 |
| 9 | 0.20 | 0 | 0.21 | 0.21 |
| 10 | 0.27 | 0.33 | 0.05 | 0 |
| 11 | 0 | 0 | 0.62 | 1.78 |
| 12 | 0 | 0 | 0 | 0 |
| 13 | 0 | 0 | 0 | 0 |
| 14 | 0.53 | 0.55 | 0 | 0 |
| 15 | 0.09 | 0.20 | 0.68 | 1.13 |
| 16 | 0.95 | 1.80 | 0.40 | ... |
| 17 | 1.55 | 1.23 | 3.30 | 6.37 |
| 18 | 0.31 | 1.20 | ... | ... |
| 19 | 0.80 | 0.26 | ... | ... |
| 20 | 0 | 0 | ... | ... |
| 21 | 0 | 0 | 6.49 | ... |
| 22 | 0 | 0 | ... | ... |
| 23 | 0 | 0 | ... | ... |
| 24 | T* | 0.54 | ... | 0.22 |
| 25 | 0.21 | 0.08 | 0.21 | 0.50 |
| 26 | 0.22 | 2.02 | 0.61 | 2.70 |
| 27 | 2.19 | 1.60 | 1.71 | 7.73 |
| 28 | 0.06 | 0.06 | 4.13 | 0.04 |
| 29 | 0.17 | 0.31 | 0 | 1.01 |
| 30 | 0 | 0 | 0.45 | 0 |
| 31 | 0 | 0 | 0 | 0 |
| Total for month | 7.56 | 10.82 | 19.20 | 22.93 |
| Average for season | 9.69 | 11.15 | 15.51 | 30.34 |

*T = Trace.

mate at 5 p.m. showed 55,400 sec.-ft. Float velocities of 25 and 30 ft. per sec. were frequently observed. These velocities were so great that soundings could not be obtained all the way across the river, it being impossible to get the weight to the bottom.

Sections of the Santa Fe trestle went out at intervals during the afternoon. The highway bridge at Oceanside was a reinforced-concrete arch bridge of six spans of about 100 ft. each, built by the unit system of construction, with three-hinged arches. This bridge seemed to be withstanding the flood, although there were some vibration and great scouring around the piers. The center pier showed some cracks on the downstream side. The river continued to rise, reaching its crest about 9:30 p.m., at an elevation of 2 ft. above that at the time the last-noted measurement was made. The remaining Santa Fe trestles had already been swept away. About this time the concrete bridge at Oceanside went out. It is impossible to state exactly how the failure occurred, it being too dark to see; but the writer believes that the center pier was undercut and washed out and that the rest of the bridge, owing to its construction, fell like a pack of cards.

Soundings made of the river showed deep depressions around each of the piers, and the scour at these points during the flood was very great. The peak of the flood was probably in excess of 70,000 sec.-ft. The entire San Luis Rey Valley was inundated (Fig. 1), the stream extending from hill to hill—a distance of probably 1½ mi.—and covering an area of over 1,000 acres. The drift along the county road on the south side of the valley indicated a 6-ft. depth of water at that point.

### Widespread and Varied Damage Caused

This valley had been used for beet culture and alfalfa. A large pumping plant erected by the South Coast Land Co. was completely lost (see Fig. 2). The pump house was a reinforced-concrete structure, and the equipment was of the highest type of modern water-works steam-driven pumps, completed about a year ago. It was located about a mile from the channel of the river, and flood-protection work of levees and willow planting had been built along the bank of the stream for a half-mile above the plant. It is probable that the enormous quantities of silt and débris carried by the stream in the first flood, together with that carried by the subsequent one, so built up and raised the old channel of the river that it was at a greater elevation than the adjacent valley lands. When the second flood came down this channel, its banks were overtopped and a new channel formed, cutting across the valley in a northeast to southwest direction, taking a long sweep to the westward at about the middle of the valley and returning to an old course near the narrows. The pumping plant was directly in the course of the new channel of the river, and the foundations of the building and pumps were undermined. All that remain visible of the plant are the domes of the water end of the pumps. It is probable that the main pumping engine may be salvaged.

The municipal pumping plant of the City of Oceanside is located in the narrows of the stream; and although the water rose to a depth of 2 ft. over the engine-room floor, the direct current of the stream did not reach the pumping plant, and no material damage occurred. It was out of commission for a couple of days, and temporary pumps had to be installed to maintain the domestic supply for the city.

All communication to and from Oceanside was interrupted by the second flood, and the city was isolated for a period of five days. All farms in the valleys of the lower river were completely destroyed and three people were drowned. The entire valley is covered with deposit of sand and silt to an average depth of at least 3 ft.—and in many places as much as 6 ft. Conditions along the upper river are not so bad. The valleys are more constricted and the stream better confined, although all crops along the bottom lands have been ruined. All the stream-flow gages and cable stations along the river have been washed out.

## Concrete-Roads the Subject of National Conference

Two years ago about 200 engineers interested in the construction of concrete roads and streets met at Chicago to discuss methods of securing the best results in this work. A second national conference on concrete-road construction was held in Chicago from Tuesday to Friday of last week, with a registered attendance of about 600.

How rapidly the concrete road is advancing in public favor is shown by statistics collected by the Association of Portland Cement Manufacturers. The total of concrete roads and streets laid previous to 1915 was about 1,600 mi. of roads and 19,000,000 sq.yd. of paving. During 1915 about 1,000 mi. of concrete road was built, and the total area of concrete street pavement laid was about 5,500,005 sq.yd.

There is every indication that the skepticism of many engineers and of the general public respecting concrete roads is rapidly passing away and that the great advantages of the properly built concrete road in durability and low cost of repair are beginning to be understood.

The chairman of the conference was Dr. W. F. M. Goss, Dean of the Engineering Schools of the University of Illinois, and the secretary was J. P. Beck, Secretary of the Association of Portland Cement Manufacturers. The conference held six sessions, at which were presented the reports of 17 different committees, each dealing with some special aspect of concrete-road work. Several papers and addresses by individuals were also presented, and most of the committee reports received extended discussion.

A committee on resolutions was appointed at the opening of the conference, made up of about a score of the engineers of most experience in concrete-road work. To this committee all the committee reports and discussions were referred, and it brought in a report at the closing session summarizing the reports as a general statement of good practice in concrete-road work. This report after some amendment was adopted by the conference.

The entire proceedings of the conference are to be edited and published and, as was remarked in the sessions, will constitute the bible of the concrete-road industry. Space permits only a brief review at this time of some of the salient facts brought forward during the conference.

**Cost of Concrete Roads**—The Committee on Cost of Constructing Concrete Roads, through its chairman, C. J. Bennett, State Highway Commissioner of Connecticut, reported that the cost per square yard of the concrete roads built in 1915 varied from 95c. to $1.28. In New

York and Ohio, in each of which states a large mileage of concrete roads was built in 1915, the average cost was 98c. and $1.02 respectively.

Cost of Annual Maintenance—Much skepticism has prevailed in the past concerning the reported low figures of maintenance cost on concrete roads. The chairman of the committee on this subject was A. H. Hinkle, Deputy Commissioner of the Ohio State Highway Department, which state has about 260 mi. of concrete roads that have been laid 1 to 5 yr. The actual figures of maintenance cost on 127 mi. of these roads for work on the concrete surface alone, including cost of superintendence and including also the entire cost of a number of tar kettles purchased during the year and still in use, was $29 per mi. Cost data of maintaining concrete state roads in Illinois show figures varying from 0.22c. per sq.yd. per annum to $0.32c., according to the kind of equipment used. This is equivalent to a cost per mile of a 15-ft. wide road of $19.36 and $28.16 per annum. In view of these and other figures the committee reported that "a conservative estimate for concrete roads 1 to 5 yr. old is a maintenance cost of less than ½c. per sq.yd. per yr." This is equivalent to less than $44 per mi. of 15-ft. road.

Details of Construction—In the reports of the committees reviewed, as well as in all the proceedings of the conference, it was urged that high quality of materials and workmanship is essential to the success of concrete roads. Twelve essentials of good practice were summarized as follows in the report of the Committee on Resolutions:

1. Drainage of the road bed is vital.
2. Subgrade must be of uniform density and should be compacted.
3. Aggregates must be clean and hard.
4. Fine aggregates (sand) should be coarse and well graded.
5. A rich mixture must be used.
6. Materials must be accurately proportioned.
7. Mixing must be most thorough.
8. Sloppy concrete must not be used
9. In general the use of reinforcement is justifiable.
10. Inspection must be intelligent and thorough.
11. Concrete must be protected from rapid drying.
12. Pavements must not be opened to traffic too soon.

Probably the most common fault in present concrete-road building is mixing the concrete too wet. In many of the committee reports it was urged that a stiff concrete be used and not a soft, mushy mass. The conference agreed after much discussion that a width of 10 ft. was desirable for single-track roads and 18 ft. for double-track roads, to give greater safety for high-speed traffic. On the other hand, many roads of 8-ft. and 16-ft. widths are being built. Stone or gravel shoulders to increase the width of narrow roads were not favored, as the high cost of maintaining them, added to their first cost, makes their total expense greater than that of an equal width of concrete that costs very little to maintain. A minimum thickness of 6 in. was recommended and a crown of $^1/_{50}$ to $^1/_{100}$ of the width.

Proposals to recommend the use of reinforcement in concrete roads were strenuously opposed in view of the fact that only a small percentage of concrete roads thus far built are reinforced. The specifications adopted, however, advise the use of transverse reinforcement on roads over 20 ft. in width, to prevent longitudinal cracking. A spacing of about 50 ft. was recommended for transverse joints. The tendency of practice is away from the use of metal joint plates.

By a resolution adopted at the closing session the conference declared that a third conference should be held and continued the advisory committee with power to fix the date and make necessary arrangements for such a conference. Copies of the proceedings of the conference just closed, when published, may be obtained from the secretary, J. P. Beck, 208 So. La Salle St., Chicago.

## American Concrete Institute Is Much Alive

Contrary to the previously held impression, the twelfth annual convention of the American Concrete Institute at Chicago, Feb. 14 to 17, did not prove the dying effort of the organization. Over 450 members and guests were registered, the nine sessions were well attended, and the board of direction was so impressed with the spirit of appreciation of the association held by those present that a proposed dissolution scheme was not only abandoned, but the program for the ensuing year was laid out with a renewed confidence, somewhat absent from the deliberations of the board during the last few years.

On account of the withdrawal of certain large contributions from the cement and allied interests the financial prospects of the coming year are not so bright. To meet the new conditions, the society is to enter into a period of rigid economy, which imposes the giving up of a paid secretary and an office, as well as a reduction in the volume of printed proceedings. At the same time Leonard C. Wason, of Boston, who was reëlected president, has offered gratis the use of his office force to conduct the necessary headquarters affairs; Walter S. Hinds, of the Atlas Portland Cement Co., New York, will serve as Secretary without compensation and will take care of the editing and the publication of the proceedings. It is hoped in the future to devote more attention to committees, so as to continue at a maximum efficiency their very valuable contributions to the knowledge of concrete.

The program of the convention was overloaded, in which respect it did not differ from the programs of previous years or, in fact, of most technical meetings. The president very wisely had set a time limit on papers and discussions; but unfortunately a strict adherence to the rule acted to cut short useful discussions, in order to admit useless papers that were on the program and therefore had the right of way. Had the pure descriptions of structures and the mathematical discussions been left from the program, more time would have been had for the practical and controversial debate that, after all, is the only matter which can be appreciated or understood on the floor of a convention.

### Committee Reports of the Institute

No committee report presented contained a specification or recommended practice that had had the required publication before the meeting, so no final action on committee reports was taken, except that the definitions submitted last year by the Committee on Nomenclature were adopted and, in a slightly altered form, sent to letter ballot. In brief, the action of the committees was as follows:

Insurance—Progress report and suggestions as to methods of collecting data. Building Blocks and Cement Products—Combination of specifications on blocks, bricks and stone into a new suggested specification and a new

specification on drain tile; ordered printed. Reinforced Concrete and Building Laws—Report on proposed revised methods and suggestions as to building work; referred back to committee. Fireproofing—Advised waiting for results of proposed tests on columns under load and fire conditions soon to be conducted at Underwriters' Laboratories. Treatment of Concrete Surfaces—Slight changes in specification and report that Bureau of Standards is conducting service tests on stucco. Sidewalks and Floors—Proposed program for future work. Concrete Roads and Pavements—Minor changes in specifications and elimination of any specific description of reinforcing methods. Concrete Aggregates—Progress report. Reinforced-Concrete Highway Bridges and Culverts—Extensive and valuable report on methods of design and construction.

### Flow of Concrete Under Early Loads

The most interesting group of papers was the one on tests, in which a generally unrecognized phenomenon was emphasized. The matter may well be abstracted in the following paragraphs from a paper by A. T. Goldbeck and E. B. Smith on "Tests of Large Reinforced-Concrete Slabs," the slabs being those recently tested by the Office of Public Roads at Washington:

> One of the important facts which these tests brought out, and which this paper seeks to impress upon concrete engineers, is that the time element is a very large and important factor in determining stress values and their distribution from the fiber deformations. Nearly all materials of construction with which the engineer has to deal will yield an approximately constant relation, independent of the time, between the load and its accompanying deformation within the limit of elasticity. This is not true in the case of concrete, which exhibits a marked flow, or molecular adjustment, under working stresses extending over time periods of several weeks. During the tests of one of these slabs in August, 1914, this phenomenon was first discovered, and a series of experiments was begun to obtain data regarding the flow of concrete. Since it is not the object of this paper to discuss the facts or the theory of this phenomenon, the following statement will be sufficient for the present purpose:
>
> When concrete is subjected to a comprehensive fiber stress of 700 lb., the immediate fiber deformation is only about 50% of what it will be if the stress is maintained for three weeks. Within 24 hr. after the application of the load the deformation has increased an additional 20%; during the first hour the deformation may show a change of 5% or more. Furthermore, the recovery of the deformations, after the removal of the load, is slow and not complete.
>
> This fact is mentioned here to show the importance of taking all the deformation readings at approximately the same time after the load is applied, and in each case a new set of deformation readings under more load must be taken just before each load application.
>
> The authors' purpose is accomplished by emphasizing that such tests, and in fact all tests for stress and deformation values in concrete, must be conducted only with a full realization of the importance of the time factor. The immediate fiber deformation is the value which should be used, and not the value obtained after a long-time suspension of the load, nor that which contains the effect of several applications and removals of loads.

This condition was emphasized in a paper by Prof. A. H. Fuller and Prof. C. C. More, University of Washington, and in the discussion of the paper by Professor McMillan, of the University of Minnesota. The tests reported show that, for a considerable time after initial setting (in some cases now nearly two years), deformation in the concrete continues. In one 10x10-ft. slab under test by Professor McMillan the deflection is constantly increasing under the designed load of 100 lb. per sq.in., but at the same time the concrete shrinkage is so great that the steel in the lower part of the slab is in compression. None of the speakers would offer any comment on the specific reference of such observations to the future integrity of concrete structures.

Prof. Duff A. Abrams, Lewis Institute, Chicago, in his paper "Tests of 885 Sands," presented some exceptionally complete analyses of tests made on sands from widely distributed localities and of varying composition. Professor Abrams has established a new constant, called the "fineness modulus," which is obtained by adding together the percentages retained on the Fuller-curve screens, and has discovered what he considers to be a comparable relation between the behavior of a sand as a fine aggregate and that modulus. His tests are on a great many specimens and cover a wide variety of conditions.

In discussion Rudolph J. Wig, Bureau of Standards, repeated what had been stated several times before on the floor, that the gradation of a sand was not an indication of its value as a concrete aggregate. He claimed that Professor Abrams' results were mostly on grand averages and that the individual sand test varied so from the average of its class as to make that average valueless. Mr. Wig's deductions are based mainly on an extensive sand-test series to be published by the Bureau of Standards.

### Design and Construction in Concrete

Flat-slab floors were treated in a remarkably clear and impartial paper by Alfred E. Lindau, Corrugated Bar Co., Chicago, entitled "Construction of Flat-Slab Floors." In it all the various systems were described. Other papers on the same subject were by Edward Smulski, Boston, Mass., who described a flat spiral system of reinforcement of his own, and by Prof. H. C. Eddy, Minneapolis, who elaborated his now well-known, though not so well understood, theory of the action of flat plates. Professor Talbot, University of Illinois, pointed out that Professor Eddy reduced the bending moment in a slab by one-half by assuming that a crossed system of embedded rods acts as a continuous metal plate and by another half because of the tensile resistance of the embedding concrete. He thus assumes that the steel in a flat slab is subject to only one-fourth of the stresses as calculated by the beam theory. Professor Talbot said that such a statement is so absurd that, coming from so eminent an authority on mathematics as Professor Eddy, it could hardly be taken seriously and must be assumed to be intended as a joke or a hoax on the profession.

Other papers on design and construction, best explained by their titles, were: "Unit Costs in Construction," by Sanford E. Thompson, Newton Highlands, Mass.; "Design of Concrete Buildings," W. P. Anderson, Ferro-Concrete Construction Co., Cincinnati, Ohio; "Design of Reinforced-Concrete Foundations for Buildings," R. L. Bertin, New York City; "Forms for Concrete Work," R. A. Sherwin, Aberthaw Construction Co., New Haven, Conn., and "Reinforced Concrete in Sewer Design," W. W. Horner, St. Louis, Mo. The paper on concrete chuting, by C. C. Brown, Indianapolis, Ind., was a statement of the fundamental principles involved in such work, but studiously avoided any reference to the absorbing question of the final strength of concrete deposited in such manner. There was no discussion.

A number of papers on completed concrete structures were included in the program, as well as a most interesting résumé by Prof. W. K. Hatt, Purdue University, of the history of reinforced concrete, as indicated by the literature of the subject prior to 1900.

# News of the Engineering World

## Engineer Wanted on Public Service Commission

House Bill No. 28, now in the Committee on Corporations of the Maryland legislature, proposes to amend the public-service law of the state by adding at the end of the third paragraph of Section 414 the following, "and one of said commission shall be an engineer." The Engineers Club of Baltimore has sent a letter to each member of the state legislature, giving numerous reasons why the amendment should be passed. The following is an extract from the letter:

> It is universally recognized that mathematics, science and engineering subjects are the greatest factors in the education of the human intellect and development of the power of differentiating substance from elusive matter. An engineer is the best-prepared person to grasp intricate and complex problems in any line involving public utilities. Any commissioner, before he is of much service as such, must be educated along these lines.
>
> The problems of the Public Service Commission are legal, financial and engineering. The greater part of those thus far presented have been engineering questions, such as the gas-rate case, the telephone case, the orders issued to the United Railways and Electric Co. relative to physical valuation, etc. The cost of transportation—in fact, the cost of anything in the public-service line—is an engineering problem. The handling or operation of the service of a public-service utility is also an engineering problem.
>
> Therefore it is advisable that at least one commissioner have the training and education that will enable him to pass on engineering problems without being entirely dependent upon the statements of interested experts.

This bill, if passed, will not become a law until June 1, 1916, so it does not affect the appointment of the commissioner whose term begins May 1. As the next term does not expire until May 1, 1918, it can readily be seen that this bill is being presented from a motive of public welfare and not to further the interests of anyone in particular.

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## United States Patent Office Force Reorganized

A bill for the reorganization of the working force in the United States Patent Office was signed by President Wilson on Feb. 15. This law is expected to increase the efficiency of the Patent Commissioner's staff by encouraging competent examiners to remain in the Government service instead of going into private employment at the time when their training has made them valuable.

By the office reorganization now authorized the number of examiners in the several grades is equalized, which will permit of faster promotion from the lower grades. Under the old organization there have been 110 fourth assistant examiners, 88 third assistants, 73 second assistants and 63 first assistants. Under the new plan there will be 86 examiners in each of these grades. With the old plan and the growth in the work of the office each primary examiner had to supervise the work of twice as many assistants as was the case 30 years ago.

The new law provides for increasing the number of assistants of higher grades in each division, giving the primary examiners the help of assistants better trained in their work. The number of examiners-in-chief is also increased from three to five. These men constitute the board of appeals of the Patent Office and have to pass upon about 1,100 cases per annum. Under the old law there was no way of assigning anybody else to aid in this work, even in case of protracted illness of members of the board, so that the lengthy illness or absence of two examiners-in-chief automatically put a stop to the hearing of appeals. The new law also increases the number of law examiners to five. Heretofore the law provided that two of the second assistant examiners might be women; the new law removes this limitation.

The effect of the reorganization of the office under the new law will be to increase the average salary of assistant examiners from $1,880 to $1,950 per annum. The law increases the number of positions in the office by 13 and adds $55,500 to the amount of the Patent Office payroll. Last year the office turned into the Treasury a surplus of $240,000.

The plan for this reorganization was worked out by Commissioner Ewing last year, and the bill then passed both houses, but was sent from the Senate to the House on the day of final adjournment, too late to be signed by the Speaker. This year the measure was placed near the top of the Senate calendar at the beginning of the present session and was soon passed by both houses.

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## Nashville Paving Work, 1914-15

Commendation of the general character of the work, criticism of the way the contracts were let and recommendations for bettering conditions in the future are features of a report on paving contracts at Nashville, Tenn., during 1914 and 1915, aggregating $1,100,000. The report was submitted to the City Commission, or Council, on Jan. 31 by James Cameron and staff, auditors, of New York City, and Walter J. Douglas, of Barclay Parsons & Klapp, consulting engineers, New York City. The report is one of a series that may be said to constitute a broad and detailed survey of the city affairs of Nashville.

As to the character of the work the report states that the construction was of high quality, as shown both by laboratory and by field tests. A peculiar feature of the paving contracts was that bidders were required to give separate figures on construction and maintenance for a five-year period, although the contract awards were based on the construction figures only. Under this plan a number of contracts were given to bidders whose combined construction and maintenance figures were higher than the total of these figures as submitted by other bidders. On the completion of the work, payment was made both for construction and for maintenance. The report urges that hereafter, in case separate bids are taken on construction and maintenance, the commissioners postpone payment for maintenance in order to insure prompt repair of pavements at any time during the guarantee period.

In the course of a somewhat lengthy discussion of contracts for bitulithic pavements the report declares

that a license provision in the specifications for this pavement practically eliminated competition. In the opinion of the authors of the report the city might have saved 15c. a yard on 156,000 sq.yd. of bitulithic pavement laid in 1914, or a total saving of $70,000 if there had been open competition for the work.

The penetration type of bituminous pavement has been too extensively used in Nashville, according to the report, in view of the fact that "it is more or less experimental." The report recommends that no more penetration pavements be laid in Nashville until those already down or contracted for have been proved satisfactory by "at least two years of service."

Prior to 1914 Nashville had comparatively little pavement, the report states. A large amount of work was started in 1914 without systematic planning, although it is to be said for the work of that year that it was done mostly on the main arteries of travel, whereas the work in 1915 was scattered at random over the city.

Nashville has 217 mi. of paved streets, 83% of which is waterbound macadam. A large part of this waterbound macadam must be covered with some kind of bituminous wearing surface. For this purpose bituminous concrete is recommended, or bitulithic if the latter can be secured at reasonable price. The heavily traveled streets, however, should be paved with asphalt upon a concrete base or else with brick or granite.

A noteworthy recommendation in the report is to the effect that hereafter all discussion between municipal officers relating to contracts be in writing.

In view of the fact that Nashville ought to undertake a considerable amount of paving work in the next ten years the report makes the pertinent suggestion that the city engineer, acting under instructions from the commission, should prepare a complete program looking a number of years ahead, elastic enough to permit changes to meet special conditions that may arise and also sufficiently elastic to take into account the wishes of property owners. For the immediate future a paving expert should be employed. Beyond and above this, all the engineering work of the city ought to be put in charge, the report states, of a Director of Public Works possessing municipal experience and preferably a member of the American Society of Civil Engineers. A minimum salary of $4,500 and appointment for a five-year term are advised. The city should also employ chemical engineers in connection with its paving work.

## State Engineering in Oklahoma

EDITORIAL CORRESPONDENCE

If engineers in the East think their efforts and work are not appreciated, they should live in Oklahoma for a while. The office of State Engineer of Oklahoma was created in 1915. On July 1 of that year, W. P. Danford was appointed Assistant State Engineer and on Aug. 10 made Acting State Engineer with the State Engineer's salary of $2,100 per annum. However, of this munificent sum of $175 per mo. he was instructed to set aside $50 per mo. to cover incidental expenses.

These incidental expenses included the salary of an assistant at $50 per mo. to register automobile licenses (for over two months) and the payment of the incidental expenses of the professor of civil engineering of the State University, who assisted in the organization of the State Highway Department. Of this $50 a mo. but $57.27 remained unexpended on Jan. 1, 1916.

These are some of the duties performed by the State Engineer in the first six months of his term: He conducted examinations (with the help of an examining board) of 88 applicants for registration as county engineers; he had to interview between 800 and 1,200 visitors, who came mostly from a distance and on business pertaining to roads and bridges, and it was incumbent upon him to give ample time to these people to explain their local conditions and problems; he had to maintain a correspondence of 10 to 15 letters a day, and there was no stenographer except one he could borrow occasionally from the state pardon clerk, frequently two weeks between borrowings; he had to travel 3,000 mi. over the state on business trips, being gone from a day to a week at a time; and he had to perform all the routine of his office as well as the engineering work, because the assistant state engineer's time was entirely taken up with registering automobile licenses. He did not even have the funds to buy office equipment and had to file his plans and profiles in the drawer of his drafting table. The only chair in his office besides his own will not stand alone.

With all these handicaps, besides examining applicants for county engineer, the State Engineer has examined and approved 250 plans for bridges, to cost $125,000, and 50 plans for culverts, to cost about $20,000. He has examined and rejected about 50 bridge and culvert plans. He has drawn up blank forms for county-engineer reports and compiled statistics of 12,604 mi. of road that has been designated as state roads; of 994 mi. of state road surveyed; of 1,050 mi. of road built and of the money expended in the different counties on roads.

The state-highway law is indefinite and deficient in many points, and the State Engineer has little real authority. The road work, as in Kansas, is wholly in the charge of the county commissioners. The only road work in which the state may directly engage is that on which convicts are employed, and even in such cases the counties pay the transportation expenses and the cost of feeding stock, the guards' board bills, supplies, power, etc.

## To Train Soldiers by Mail

Governor Whitman has launched a project for organizing the engineers in the employ of New York State into a military reserve corps. The invitation to enroll has been extended through State Highway Commissioner Duffey, and it is expected that about 1,200 men will be available for training. It is planned to give correspondence courses in military theory and to supplement them by periods of intensive drill and practice in the regular-army instruction camps such as the one started at Plattsburg, N. Y., last summer.

## Extensive Road Improvements

A $3,000,000 bond issue for permanent road improvements in St. Louis County, Missouri, was authorized by a large popular majority on Feb. 15. The county has had a system of macadam roads, which for a number of years have been kept in good repair and oiled. These roads get hard service from the City of St. Louis.

The cost of maintenance is now so high that it is considered a measure of economy to construct more permanent road surfaces. It is estimated that the $3,000,000 voted will be enough to reconstruct all the main arteries leading out from St. Louis.

**A Rear Collision** on the New York, New Haven & Hartford R.R. at Milford, Conn., at about 11:30 a.m., Feb. 22, killed 10 persons and injured 30 others. A westbound local passenger train crashed into the rear of a westbound express stalled ahead of it, and the two trains, jumping from the tracks, were sideswiped by a freight train moving west on the adjacent track. The local was steam operated, and the boiler of its locomotive burst. The railroad company states that all signals were properly set. Survivors allege that they heard a steam whistle blown for 1½ min. just prior to the collision and believe that it could only have been sounded by the freight in an endeavor to warn the express. Examination showed the brakes on the local to be in good order and that they had been set hard. The engineer of the local was killed, as was also the flagman of the express. Most of the other dead and injured were in the rear coach (steel) of the express.

**Air Blowout in Subaqueous Tunnel Work**—A blowout occurred at 4 p.m. on Feb. 19 in the north tunnel, east end, of the South Ferry-Montague St. subway tunnels now building under the East River, New York City. Two parallel tunnels are being driven by means of shields, and the work has been pushed out under the East River a distance of about 600 ft. from the foot of Montague St., Brooklyn. The shield in the north heading was about to be moved forward its 26-in. lift and, to this end, the shoring in the excavation was being removed by two sandhogs, when a hole began to form in the roof of the excavation in front of the shield. A workman attempted to plug the hole with a bag of sand, but was drawn up and into the river. Immediately following, the two who were removing the shoring were drawn up and out. One of the latter men survived. The excavation apparently had proceeded under a pocket or depression in the mud, so that the mud roof had thinned, reducing the resistance against the 24-lb. air pressure maintained at that time. As the hole enlarged, water entered the tunnel until it was flooded. Repair work started on Feb. 21, by dumping clay over the break. A blowout occurred on Nov. 30, 1915, in the south tunnel, west end, of that other pair of tunnels which completes the East River connections between the New York and Brooklyn subways now under construction. This latter pair connects Old Slip, New York City, with Clark St., Brooklyn. The Flynn-O'Rourke Co. is contractor for both jobs (four tunnels).

**Smoke-Prevention Education**—As a part of the campaign of the New York City Department of Health, the Bureau of Public Health Education, C. F. Bolduan, M. D., Director, has issued a pamphlet for steamboat owners, describing how the firemen can save coal and keep the air clean.

**A Rock-Road Bond Issue of $1,300,000** was defeated by the voters of Saline County, Missouri, on Jan. 19 by a vote of 2,969 against to 2,310 for. A total of 192 mi. of improved roads was proposed. It is stated that the defect was chiefly due to "a nasty municipal fight in Marshall, the county seat and principal town."

**Kansas City, Mo.**—Bonds to the amount of $475,000, payable in 20 yr. at rates of 4 and 4½% annually, have been sold to St. Louis and Boston bankers at a premium of $12,680, or about 3%. Bonds to the amount of $25,000 in denominations of $100 brought bids from but two individual investors. The proceeds of the bond issue are to be used for the construction of buildings for the general hospital, the fire and police departments and the grading of the park in front of the Union station.

**The Irrigation Canal System** of the Imperial Valley, California, was bought at auction on Feb. 8 by the Southern Pacific Co. for $3,875,000. This is a step in the transfer of the property of the California Development Co. to the Imperial Irrigation District. It is expected that the transfer will be completed soon. Col. William H. Holabird is receiver, and Chester Allison is chief engineer for the California Development Co. C. R. Rockwood, El Centro, Calif., is chief engineer of the Imperial Irrigation District.

**The Can and Platform-Wagon** system of garbage collection has just been adopted for Utica, N. Y., by Commissioner of Public Safety, H. R. Hayes. Eight wagons and about 700 cans will be required by the contractor, J. R. Baxter. The house accumulations are transferred to the wagons as before; each can is cleaned and sterilized every time it goes to the reduction plant. The contractor receives $2,178 per mo. and the collection contract has just been signed for 5 yr. At present Utica has a population of about 85,000. The contractor for the reduction of the garbage is Henry Steppenbach. A fertilizer plant is maintained, the most of the grease being used as fuel and a little sold for soap. Some 50 to 60 tons a day are received, the contractor getting $4,097 per year. Ashes are collected and dumped for $19,560 per year, the contractor (Utica Carting Co.) buying property for dumps and selling it after the fills are completed.

**All the Pump-Building Plants** of the International Steam Pump Co. are to be sold under foreclosure to the highest bidder, under decree of the United States District Court for the Southern District of New York. The property is to be sold in 15 different parcels, several of which are so arranged as to include one principal works of the company. Among the principal plants to be sold are the Deane Steam Pump works at Holyoke, Mass.; the Laidlaw-Dunn-Gordon Pump works at Hamilton, Ohio; the Snow Steam Pump plant at Buffalo, N. Y., and the Power and Mining Machinery plant at Cudahy, Wis. Other parcels include stock in the Henry R. Worthington Co., the Blake & Knowles Steam Pump works, the Holly Manufacturing Co., the Fred M. Prescott Steam Pump works, the Denver Rock Drill and Machinery Co., the International Gas Engine Co., and the Jeanesville Iron Works Co. The dates of sale of the different parcels range from Feb. 25 to Mar. 7. The sale is made in accordance with the plan adopted for the reorganization of the company.

**Prizes for the Best Designs** for workmen's houses amounting to $2,100 are offered by the "Roosevelt America-First Fund." The prizes are offered in connection with the movement for better housing of newly arrived immigrants in America who are employed in connection with industrial operations. The competition is divided into two classes. Those competing in the first class should submit designs for the following: (a) Single-family house; (b) combined family and lodging house; (c) boarding house or community dwelling. The prizes offered in this class are a first prize of $1,000; second prize, $500; third, fourth and fifth prizes of $100 each. In the second class a first prize of $200 and a second prize of $100 is offered for the best design for a boarding car for use of a railroad construction gang. A public exhibition will be held of the prize-winning plans and other plans selected by the jury of awards. The contest is to close on May 1, 1916. Among the members of the committee in charge of the competition are Frank Trumbull, Chairman of the Board of Directors of the Chesapeake & Ohio R.R.; Felix M. Warburg of the Federal Reserve Board, and George A. Cullen, of the Delaware, Lackawanna & Western R.R. Further information regarding the conditions of the contest may be obtained from the "Immigrants in America Review," 20 W. 34th St., New York City.

**Engineers and State Engineering in Minnesota**—An engineering staff connected with the university-extension division of Minnesota State University, to furnish information and advice to municipalities in regard to public improvements contemplated, has been advocated by the League of Minnesota Municipalities, presumably on lines already followed in Wisconsin. Such a move would require authority and appropriation by the state legislature. It appears, however, that the sanitary division of the Minnesota State Board of Health already deals with matters of water-supply, sewerage and sewage disposal, while the State Highway Commission reports upon bridges and roads. Therefore, paving is the only important class of improvements not already covered, and most cities interested in paving employ city engineers.

These conditions were reviewed in a report presented at the annual meeting of the Minnesota Society of Surveyors and Engineers in St. Paul on Feb. 10. It was shown that the proposed engineering staff would involve a duplication of efforts, and the Society adopted a resolution opposing the creation of such a staff. It also recommended that state departments should refrain from indorsing or recommending engineers for proposed work, the information given by such departments in response to inquiries being limited to the names of municipalities where similar work has been done. This course would leave the inquirers to investigate for themselves the character of the service rendered and the qualifications of the engineers available.

## PERSONALS

Mr. Glenn P. Beach, Principal Assistant Engineer of the Minnesota Railroad and Warehouse Commission has resigned to engage in other business.

Dr. Eliot Blackwelder, Professor of Geology at the University of Wisconsin, has been appointed Head of the De-

partment of Geology at the University of Illinois. Dr. Blackwelder was paleontologist with the Carnegie Institute expedition in China in 1903. He has been a member of the Wisconsin faculty since 1904.

Mr. W. W. Cunningham, for the past 13 years President and General Manager of the Pennsylvania Clay Co., of Pittsburgh, Penn., has resigned his position and is now associated with the Clydesdale Brick and Stone Co., Oliver Building, Pittsburgh, as Vice-President. He will have charge of the manufacture and sales of the "Clydesdale" shale paving block that will shortly be put on the market.

Mr. John T. Hopkins, Assistant Engineer, of Oswego, N. Y., has been appointed Borough Engineer of Highland Park, N. J. He will virtually be borough manager; but as there is no statutory warrant for appointing a borough or city manager in New Jersey, the legal responsibility for his work other than as borough engineer will rest on the mayor and council. Besides the legal compensation for serving as borough engineer Mr. Hopkins will be paid a further sum, provided by public-spirited citizens who desire to see the manager idea given a trial. Mr. Hopkins was one of about 200 men who responded to an invitation for applications for this position which was printed in "Engineering News."

Mr. D. W. Cole, M. Am. Soc. C. E., who has been in the United States Reclamation Service for nearly 12 years, recently in charge of the Truckee-Carson project and construction of the Lahonton dam in Nevada, has been designated Senior Engineer and resident Manager of the Boise project, succeeding F. E. Weymouth, M. Am. Soc. C. E., who was recently appointed Chief of Construction for the Service at Denver, Colo. He will have general oversight of all projects in the States of Washington Oregon, Idaho, Nevada and Northern California, for the purpose of advising the Chief of Construction upon all reclamation affairs in that territory. His headquarters will be at Boise, Idaho.

Mr. P. P. Borden, M. Can. Soc. C. E., of Montreal, has been appointed a Member of the Board of Engineers of the Quebec Bridge, filling the place made vacant by the death of C. C. Schneider. Mr. Borden is a graduate of McGill University and for several years after graduation was assistant engineer in the bridge department of the Canadian Pacific Ry. From 1904 to 1906 he was Assistant Chief Engineer, structural department, Montreal works of the American Locomotive Co. From then until 1908 he was Structural Engineer of the Canadian Pacific Ry. In 1908 he became Assistant Engineer, Board of Engineers, Quebec Bridge, and for the last three years has been Assistant to the Chief Engineer, C. N. Monserrat.

## OBITUARY

Alexander Saunders, President of D. Saunders' Sons, Inc., of Yonkers, N. Y., manufacturers of pipe machinery, died at his home in that city, on Feb. 1, in his seventy-eighth year.

William Evarts Tracy, recently on the engineering staff of the Anaconda Copper Co., at Helena, Mont., and for 10 years previous to that connection Engineer of the Liberty Bell Mining Co., at Telluride, Colo., died at Helena recently, at 38 years of age. He was born in Plainfield, N. J., and was graduated from Yale University in 1900, after which he attended the Columbia School of Mines.

Louis Duncan, Past-President of the American Institute of Electrical Engineers, a member of the firm of Duncan, Young & Co., Consulting Engineers of New York City, died at his home in Pelham Manor, N. Y., on Feb. 13, at 54 years of age. Dr. Duncan was born in Washington, D. C. He was graduated from Annapolis in 1880 and three years later was sent by the United States Government to Johns Hopkins University to take a special course in electricity and physics. He received the degree of doctor of philosophy from Johns Hopkins in 1887 and resigned at that time from the navy to become a member of the Johns Hopkins University faculty. He established the course of electrical engineering of that university and was head of the department of electrical engineering until 1898. During that time he became associated with Frank J. Sprague and Cary T. Hutchinson and acted as consulting engineer on the electrification of the Baltimore tunnels of the Baltimore & Ohio R.R. and the equipment of the Third Ave. elevated-railroad system of New York City, later being consulting engineer also of the New York City Rapid Transit Commission. During the Spanish-American War he became major of the First Volunteer Engineers. In 1899 Dr. Duncan inaugurated the electrical-engineering course at the Massachusetts Institute of Technology and was head of the department of electrical engineering from 1902 to 1904. He was twice elected president of the American Institute of Electrical Engineers (1895 to 1897). Dr. Duncan was a writer of numerous articles on engineering subjects contributed to the technical press of this country.

## ENGINEERING SOCIETIES

COMING MEETINGS

AMERICAN ROAD BUILDERS' ASSOCIATION.
Feb. 28 to Mar. 3. Annual convention in Pittsburgh. Secy., E. L. Powers, 150 Nasau St., New York City.

ENGINEERING SOCIETY OF WISCONSIN.
Feb. 25-26. Annual meeting in Madison. Secy., L. S. Smith, Madison, Wis.

**Kentucky County Road Engineers' Association** held a convention in Lexington on Feb. 4 and 5. There were four formal addresses pertaining to road construction and a number of short talks.

**North Dakota Society of Engineers** is holding its eighth annual convention, Feb. 24 and 25, at Grand Forks. The program promises sessions of unusual interest. The secretary is E. F. Chandler, University.

**The Florida Drainage Association** will hold its third annual meeting at Sanford, Fla., Feb. 28 and 29. In addition to various addresses on the subject of land reclamation for agricultural and other purposes there will be an exhibition of ditching, leveeing and pumping machinery. For further information address Benjamin Getzoff, chairman of Committee of Arrangements, Bartow, Fla.

**American Building Show** has been going on in Cleveland since Feb. 16 and will continue through Feb. 26. In the first week of the show the annual convention of the National Builders' Supply Association was held. This week the National Brick Manufacturers' Association is in session, as are also the National Paving Brick Manufacturers' Association, the American Ceramic Society, the Hollow Tile Manufacturers' Association and the Ohio Association of Lumber Dealers. The daily attendance at the "First American Complete Building Show" is estimated to average 10,000.

**San Francisco Association** of members of the American Society of Civil Engineers at its eleventh annual meeting elected H. L. Haehl, president and Jerome Newman vice-president. A communication was read from Mr. Newman, suggesting that an effort be made to ascertain the views of the association as to the extent to which it is justifiable for one engineer to use the plans and specifications of another, without permission and without giving credit to the latter. A committee of five was appointed to consider and define principles of ethics in engineering practice and report at a subsequent meeting.

**A Snow-Removal Conference** will be held on the evening of Mar. 8 in New York City in the assembly hall of the Automobile Club of America, 247 West 54th St., under the joint auspices of the National Highway Association, the Automobile Club of America, the Citizens Street-Traffic Committee of Greater New York and the graduate course in highway engineering, of Columbia University. The conference will open at 8:30 with an illustrated lecture on "Modern Methods of Snow Removal" by Wm. H. Connell, of Philadelphia. The proceedings of the conference will be published by the National Highways Association, Charles Henry Davis, president.

**Boston Society of Civil Engineers** held its seventh annual dinner jointly with the Boston members of the American Society of Mechanical Engineers and the American Institute of Electrical Engineers at the Boston City Club on Feb. 8. More than 500 members of the three societies were present. Charles L. Edgar presided as toastmaster, and the speakers were Governor McCall, of Massachusetts, Col. W. E. Craighill, Corps of Engineers, U. S. A., William Barclay Parsons, John R. Freeman and Dr. John A. Brashear. Governor McCall spoke of the need of engineers in the economic and efficient administration of the government and on the desirability of reducing a large number of commissions in Massachusetts. Colonel Craighill spoke of the work of military engineers in the time of war and the plan of the War Department to form a reserve corps of engineers. He described briefly the work that this reserve corps would be called on to do for the defense of Boston. Mr. Freeman spoke of his recent trip to investigate the slides in the Panama Canal.

**Building Officials' Conference** held its second annual meeting on Feb. 14, at the Auditorium Hotel, Chicago. About 30 representatives of building departments and bureaus throughout the country were present. The morning and part of the

afternoon sessions were devoted to discussing the recent changes and improvements in building codes and the desirability of giving building officials discretionary power, a large part of that discussion being devoted to the question of the certificate of occupancy. Other topics that occupied some of the time were "The Use of Terra-Cotta Blocks and Gypsum Products" and "Metal Lath and Wood Shingles." The discussion on these matters proved so interesting that an evening session had to be arranged for in addition to those scheduled. The following important resolutions were adopted by the conference:

Resolved, That discretionary power should be vested in building officials to vary or modify the law, where practical difficulties arise in its literal enforcement, so that the intent of the law is secured; and that in case the building official declines to exercise such discretionary power, an appeal should lie to a board of technical experts.

Resolved, That proper means of egress from every building should be provided in the form of inclosed stairways; that outside metal fire escapes should not be considered as a required means of egress from new buildings, and that their use should be confined to existing buildings where other means of egress are impractical; and that wherever such fire escapes are used, the opening thereon should be protected by metal sash and wire glass or by fire-resisting doors with proper fusible links.

The meetings of this conference have up to this time been informal in character. It was decided, however, to form a more permanent organization. Active membership will be open to any department or bureau administering laws or ordinances affecting the safety of buildings, with respect to their construction, fire hazard, or sanitation, or to individuals actively connected with such departments or bureaus. It is intended to hold an annual meeting each winter, the meetings to be held in different cities and so far as possible in connection with some exposition or meeting of some other organization having somewhat similar interests.

Rudolph P. Miller, of New York City, was elected to serve as chairman of the conference for 1916, and James G. Houghton, Building Inspector of Minneapolis, was elected vice-chairman. Sidney J. Williams, State Building Inspector of Wisconsin, was elected secretary. Superintendent Carlin, of Brooklyn, N. Y., and Building Commissioner Forbush, of Newton, Mass., were elected members of the executive committee.

## Appliances and Materials

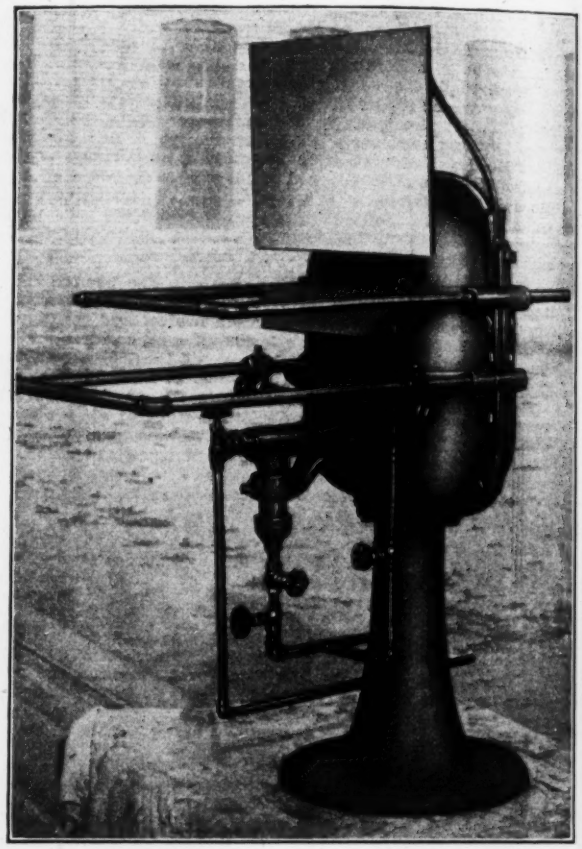
### Vitrified Segmental Culvert Blocks

Vitrified segmental blocks, which have been quite extensively used the last two or three years in sewer construction, are being introduced for highway culverts, particularly in the clay country of the Middle West. The accompanying illustration shows an 84-in. culvert in Minnesota built of these blocks. A 24-in. by 40-ft. culvert in Kansas was built by two inexperienced men and a helper in one working day's time. The arch centers used for this work were merely three templets cut to a 24-in. radius and wedged up on 4x4-in. sticks placed across the culvert just below the springing line. The templets were covered with 1x4-in. strips. On completion of a section of the arch the wedges and centering were removed and backfilling proceded at once, without waiting for the mortar to set. Adjacent blocks are dovetailed to prevent their being displaced. These blocks are manufactured by the W. S. Dickey Clay Manufacturing Co., Kansas City, Mo.

LARGE HIGHWAY CULVERT OF SEGMENTAL BLOCKS

### Oil-Burning Furnace with Preheater

The No. 3 "Leyner" oil-burning furnace, which is used extensively for heating drill steel, is now made with a preheating chamber designed to increase the capacity of the burner. The preheater is a section that fits between the body and the cover of the furnace, and it is a simple matter to attach the section to an old furnace, the addition requiring only the changing of a few bolts. The manufacturer, Ingersoll-Rand Co., claims that by the addition of the preheating feature the space for heating is doubled, the heating capacity of the furnace is increased about 50% and the efficiency to a corresponding extent.



NEW "LEYNER" NO. 3 PREHEATING FURNACE FOR HEATING DRILL STEELS

The type of burner now furnished is suitable either for high- or low-pressure air. For high pressure it has merely to be throttled. The furnace burns petroleum or any of its oils. The floor space occupied by the No. 3 furnace is 3x4 ft., and no foundation is required.

* * *

### Dump Buckets Equipped With Wheels

Wheeled skips, or dump buckets, developed for use in removing subway spoil from the various shafts on Broadway, New York City, are manufactured by the G. L. Stuebner Iron Works, Long Island City, N. Y. Being equipped with wheels, they do not have to be conveyed by cars from the foot of the shaft to the point of loading. The bucket is designed so that it will not tip while being hoisted, although when it is desired to dump the bucket a trip line is used. The hosting handle carries two latches, one on each side, which engage with the trunnions of the skip and cannot be released until the bucket is set down.

* * *

### Weir with Flow Proportional to Head

A weir of special shape, the flow over which is directly proportional to the head on it, has been devised by H. C. Alger, of the Kennicott Co., Chicago Heights, Ill. All cams and complicated mechanisms in the recording device of a water meter using this weir are eliminated, and a simple recorder for pressure head is calibrated to read flow directly. The curve of the Alger weir was obtained by experimentation and found to be a right cycloid. A line of "Cycloid Weir Meters" is being made by the Kennicott Co.

# Engineering News

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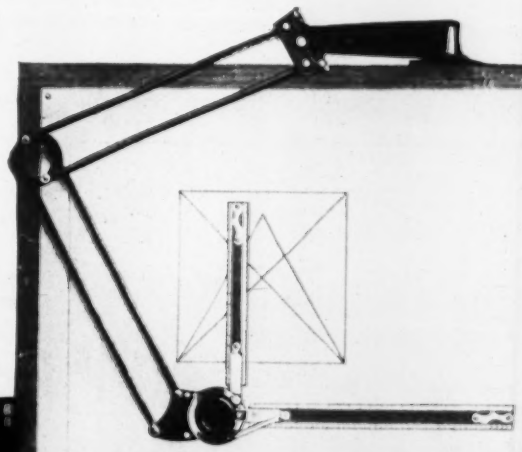
## The UNIVERSAL

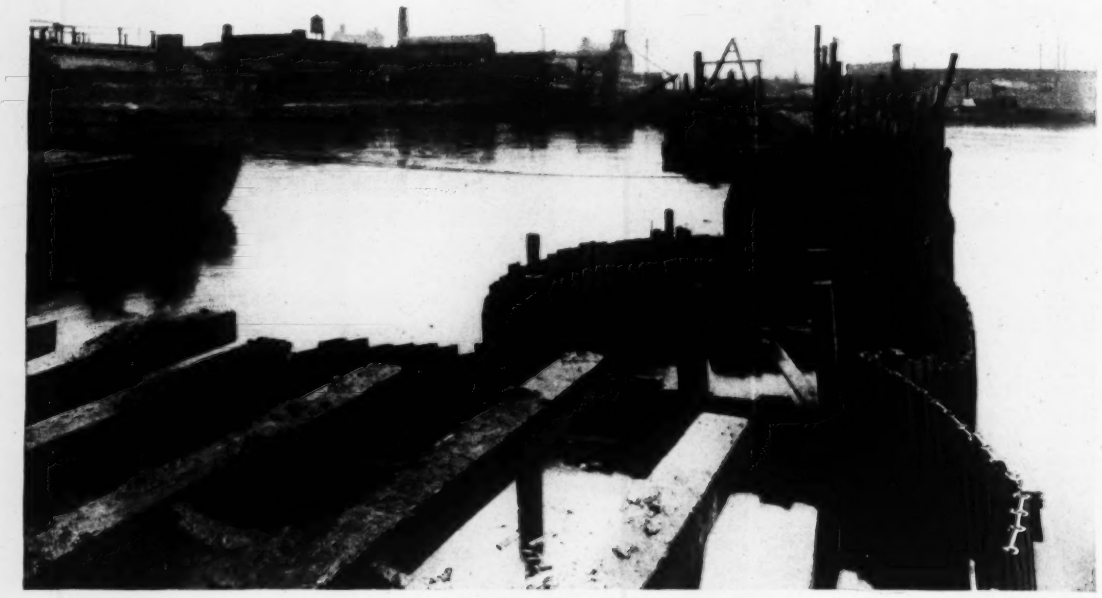
### Ball Bearing Drafting Machine is Practically Ideal

This Universal Drafting Machine is a perfected product. Among its special features are—Ball Bearing Joints, Tubular Rods, Every Working Part Adjustable, Parts Interchangeable, Interchangeable Protractors and Interchangeable Scales. It is practically troubleproof—a machine of extreme accuracy, easy, sensitive motion and great rapidity. Write for catalog giving detailed information.

**The accompanying view of the Link-Belt Co.'s drafting room shows some of the 100 Universal Drafting Machines used by this concern.**



UNIVERSAL DRAFTING MACHINE COMPANY
CLEVELAND, OHIO, U. S. A.



# Pockets on the River Side of the Lackawanna Steel Sheet Piling Cofferdam, 46th Street Piers, N. Y.

All steel sheet piling here was of the $12\frac{3}{4}$ by $\frac{3}{8}$-in. Lackawanna 35-pound Straight-Web Section, in pieces of a maximum length of 73 feet, for a penetration of 34 feet and projection of $8\frac{1}{2}$ to 9 feet above mean low water level.

The steel sheet piling driven in that part of the cofferdam which was in pocket form, was assembled around wooden templates, each fastened to four wooden piles (see smaller picture.) Assembling was accomplished by the 100-foot spliced and trussed wooden boom of a floating derrick.

The sheet piling sections usually settled under their own weight alone to within about ten feet of the rock surface, and were then driven in sets of two or three to moderate refusal, after which they were driven singly to final refusal by Monarch No. 2, Hamilton and Arnott steam hammers.

Soft material tubbed out from inside the cofferdam was placed within the pockets to displace water and to render the sheet piling joints watertight. Heavy filling was also deposited within the pockets to help in resisting thrust, and this additional filling was brought in from shore by wagons over a plank roadway extending over the pockets as fast as the filling progressed. Stringers for extension of this roadway are shown in the left hand corner of the upper picture.

The progressive steps in making this cofferdam stable will be explained in next week's advertisement.

And steps in other successful cofferdam work are explained in our book "Lackawanna Steel Sheet Piling," which will be sent to any contracting or managing engineer.

## LACKAWANNA STEEL COMPANY

**General Sales Office and Works: Lackawanna, N. Y.**

NEW YORK PHILADELPHIA CHICAGO ATLANTA
BOSTON CLEVELAND DETROIT SAN FRANCISCO
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For Great Britain and British Colonies in the Eastern Hemisphere, Cargo-Fleet-Iron Co., Ltd., Middlesborough, England. For France, Italy, Spain, French Colonies and Protectorates, Italian Colonies and Spanish Colonies in the Eastern Hemisphere: Cie Des Forges & Aciéries de la Marine et d'Homécourt, Paris, France. 228

Engineering News

VOLUME 75 MARCH 2, 1916 NUMBER 9

# Galveston Storm Repairs

By N. T. Blackburn*

*SYNOPSIS—Damages to engineering structures caused by the Galveston storm and flood of Aug. 16 and 17, 1915, are outlined. Progress on repairs to the sea wall and its sand backing, to the boulevard, and the water-supply conduits from the mainland are reviewed.*

The West Indies hurricane that passed over Galveston, Tex., Aug. 16 and 17, 1915, was in some respects of greater violence and intensity than any other in the history of the place. While the greatest wind velocity was less than in 1900, the period of high winds was greater, and the water backed up on the island from Galveston Bay remained longer. It was due to this fact that oak trees 40 and 50 years old, which had survived many storms and withstood the grade-raising operations, were killed. There is no doubt in the mind of anyone who went through the storm that the sea wall saved the city. The only damage done to the concrete of the sea wall was the knocking off of two small chips of about 2 cu.ft. capacity each from the top of the wall at the foot of 39th St., when the four-masted schooner "Dora Allison" was dashed on it during the height of the storm. This schooner, which was bound from Progreso, Mex., to Mobile, Ala., loaded with sisal, encountered the hurricane in the Gulf and was blown off her course. When she struck the sea wall, she was dragging two anchors. These caught in front of the wall, or under its toe, and the schooner fetched up and pounded to pieces on top of the wall, the fragments of hull, masts, cargo, etc., being scattered over and behind the Fort Crockett military reservation. This occurrence is considered a rather remarkable test of the sea wall.

The riprap apron in front of the wall was undermined in many places and dropped, allowing the sand beach to be scoured away (for a depth of 3 ft. in the extreme case) under the toe of the wall, thus exposing the sheet-pile foundation to the action of the teredo.

The first efforts made after the storm, both on the part of the county and the United States Engineer Office, were directed toward temporary repairs, only, to their respective portions of the sea wall. Such small riprap

*United States Assistant Engineer, Galveston, Tex.

FIG. 1. TEMPORARY RIPRAP, SAND AND SHELL PROTECTION FOR PORTION OF GALVESTON SEA WALL

as could be gathered up, broken concrete, sand and shell were placed in the front of the wall, to fill in the holes and protect the sheet-piles from the teredo (Fig. 1). These temporary repairs have now been completed, and the county is carefully placing about 24,000 tons of additional riprap as an apron, to give permanent protection along the toe of the wall.

The boulevard and sidewalk behind the sea wall were more or less damaged along their entire length. The worst damage was from 39th to 43rd St. in the Fort Crockett military reservation, and from about 6th to 19th St. in the city, where the boulevard and sidewalk were nearly or quite washed away, and with them the sand-fill behind the wall, to a maximum depth of about 10 ft. This sand was washed several blocks back from the wall, filling streets and yards. In this latter area many houses were undermined and tilted, or turned over.

The repairs in the city up to the middle of January, 1916, have been confined to straightening up houses, gathering brick from the damaged boulevard for relaying, repairing the smaller breaks in the boulevard and preparing to begin construction of a concrete sheet-pile bulkhead behind the sea wall under the new plan. The new plan consists briefly in placing a boulevard, or roadway, 100 ft. wide back of the sea wall, beginning at 6th St. where the wall turns back across the island. Sixteen feet of this roadway will be a sidewalk, and the back line of the boulevard will be El. +19, or 2 ft. above the top of the sea wall. The boulevard is to be flush with the top of the wall, with an interrupted curb 4 in. high between them, to allow free drainage of water from the boulevard back over the sea wall. Along the back side of the boulevard a continuous line of reinforced-concrete sheet piles 6 in. thick and 14 ft. long is to be placed, capped with concrete coping and tied back at 10-ft. intervals with iron rods to an 8x12-in. continuous concrete deadman, buried under the pavement 20 ft. from the sheet piles.

Along the wharf front the damage consisted principally in the scattering of ships, dredges, barges and boats of all kinds around Galveston Bay. One ship grounded in the railroad yards just west of the city; two on the mainland south of Texas City; two on the dredge spoil-bank on the north side of Galveston Channel; two on Pelican Island, in Galveston Bay; and one on Red Fish Reef, some 12 mi. up Galveston Bay from her anchorage. These ships have all been dredged off and repaired. Most of the smaller boats, barges, dredges, etc., have also been floated and repaired. Some will never be recovered.

The damage to the wharf system itself was confined principally to the entire or partial wrecking of wooden warehouse sheds on the wharves. One steel elevator conveyor was blown down and wrecked, the banana conveyors of the United Fruit Co. were twisted and damaged, and the tracks, wharves, etc., were littered with wreckage of every description. Repairs to all this damage have been nearly completed and the wreckage cleaned up.

### Restoring Railroad Connection and Water-Supply

Probably the most serious blow to the city and the railroads was the wrecking of the causeway across West Galveston Bay, cutting off communication with the mainland and incidentally destroying the fresh-water main that was laid from the source of supply on the mainland. The earth portions of the causeway, which lay on each side of the central arch- and lift-bridge portion and which consisted of a sand-mound, or fill, between parallel rows of concrete sheet piles at the toes, with concrete slabs on the sloping sides, were washed down. The railroad and interurban tracks and the duplicate water main were carried away also, and the county highway was destroyed.

FIG. 2. DRIVING LAST BENT FOR TEMPORARY RAILROAD TRESTLE OVER WRECKED PORTION OF GALVESTON CAUSEWAY

Within two weeks after the storm the Santa Fe and Southern Pacific railroads, working in conjunction, had built a trestle (Fig. 2) across the gaps and reëstablished railroad communication between the city and the mainland, the first train passing over the trestle Sept. 1.

As a fresh-water supply was imperatively needed, an 8-in. temporary main was laid for the city by the railroad forces engaged on the building of the trestle. This main is shown at the left-hand side of Fig. 2. It covered only the wrecked part of the main, that part over the concrete arches being undamaged. Efforts were then directed by the city toward repairing the broken main. This work has finally been completed, and the city is now in possession of its regular fresh-water supply.

As the old water main, which was buried in the bed of the bay alongside the causeway and was in use before the building of that structure, was undermined and also wrecked during the storm, plans have been made for a

30-in. emergency main to be laid from 4 to 8 ft. below the present bed of the bay. The bottom of this main will be 17.8 ft. below mean low tide in the middle of the bay opposite the lift span of the causeway, sloping up to about 5 ft. below the same datum at each shore line, where it will connect with the old main.

A bond issue providing funds for this project was carried in a special election on Jan. 18. It is understood that with the rebuilding of the causeway the duplicate main in this structure will be replaced. It is believed that with these mains and the emergency main in existence all danger of an interruption of the water-supply due to a storm will be averted.

### Plans for Rebuilding Causeway to Mainland

Plans for the rebuilding of the causeway are not yet decided on. The structure was owned jointly by the County of Galveston, the steam railroads and the interurban electric road entering the city. Engineers representing these various interests have met and are meeting together frequently to adjust their differences of plan and agree on a plan of rebuilding acceptable to all. While it at first appeared that there was a deadlock, the last few meetings have indicated that differences are rapidly being adjusted, as all parties are making concessions and it now appears that a plan will soon be agreed upon. When finally agreed on the plan will be submitted to the general managers of the railroads and the interurban and the county representative for ratification.

The county favors the plan of extending the concrete arches entirely across the bay, and this plan is strongly urged by the Citizens Committee of Galveston. As the preliminary estimate for the extension of the arches was about $2,000,000 a plan was at first considered of replacing the wrecked part by an open creosoted-pile trestle, the county highway being on an independent trestle with ballasted deck.

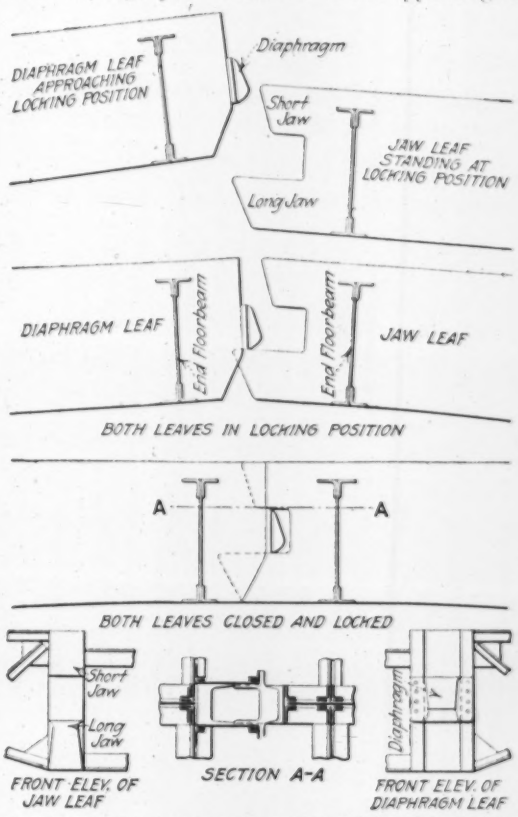
Another plan was to build a reinforced-concrete pile trestle. It is understood that on reëstimate and more thorough consideration it was believed by the designing engineer of the old causeway, William Mueser, who has been acting as consulting engineer for the county, that the arches could be extended to shore on each side for $1,500,000 by using shorter spans than the present arch-bridge part and filling in between the concrete sheet-pile bulkheads along the toe of the earth portion (which are the only parts standing of this portion) so as to make dry land on which to work in constructing the arches.

As these pile bulkheads project above the surface of the water about 4 ft. and extend from the shore out they form obstructions and restrict the free movement of the water. One of the remaining points to be settled is whether any part of these bulkheads shall be torn out so as to give more free passageway for the water, the railroad engineers favoring this for at least a part of the bulkheads. There seems to be general agreement that an open structure allowing free movement of the bay water is essential during storms and that no part of the earth portion shall be rebuilt in its old form. The question of plan is largely a financial one, but the indications are for the plan of extending the arches entirely across the bay.

The plans under discussion provide for a causeway 66 ft. in width, conforming to the arches now standing. Of this width the interurban company asks for 15 ft., the county 22 ft. and the four steam railroads 29 ft.

## Bascule-Bridge End Lock

In the recent bascule bridges built by the Scherzer Rolling-Lift Bridge Co., of Chicago, the use of moving end locks and their mechanism is dispensed with. As the leaves come together in closing, the ends of the girders of the opposite leaves are connected and locked automatically by means of what is termed a shear lock. This consists of a diaphragm on the end of one girder which engages with a recess formed between an upper (short) and lower (long) jaw on the end of the opposite girder.




AUTOMATIC END LOCK FOR SCHERZER ROLLING-LIFT BASCULE BRIDGES

In closing the bridge the leaf with jaw is lowered to such a position that the shorter (upper) jaw will be cleared by the diaphragm on the other leaf. The leaf with diaphragm is then lowered until the diaphragm engages the longer (lower) jaw, after which both leaves are lowered to their final position. This last motion, due to the longitudinal movement or advance of the leaves in their rolling travel, brings the parts into their full locked position, with the diaphragm between the upper and lower jaws.

In opening the bridge both leaves are raised simultaneously, disengaging the lock by the longitudinal movement due to the leaves rolling backward as they swing up. Dials in the operating cabin show the positions of the leaves at all times, so that it is an easy matter for the operator to control the movements. The diaphragms and jaws are steel castings.

With this construction no tail locks are required. The shear locks and track-plate bearings prevent the leaves from being lowered below their normal position. Bumping blocks which engage the counterweighted tail ends of the leaves check the travel at the full open position.

# Building an Unusual Foundation for a Heavy Machine

By G. L. Knight*

*SYNOPSIS—A spare mass foundation, useless for plant extension, owing to rapid advance in turbine design, had to be cut out. No room for new mass foundation, so a steel platform was erected on concrete-and-pile footings. Condensing-water tunnels interfered and one had to be bridged, so that heavy load would not come on it. Hollow piles driven by drop hammer; 720 hr. to drive 23 piles. Piles tested to 40 tons.*

In providing for the installation of a 22,000-kw. turbo-generator at the Gold St. generating station of the Edison Electric Illuminating Co., of Brooklyn, N. Y., some very unusual conditions and difficulties were encountered which called for exceptional methods in overcoming them. The turbine is the last of six units in the station and is No. 3 in a row from the East River end of the building.

In 1911, when the adjacent 12,000-kw. vertical unit No. 2 was installed, it was intended to make No. 3 a duplicate of No. 2 and install it during the following year or not later than 1913. In view of this plan, the foundations for No. 3, below the basement-floor level, together with the connections to the condensing tunnels, were put in at the same time as those of No. 2. Conditions of load, however, made is unnecessary to purchase additional capacity until 1914. In the meantime steam-turbine design had undergone rapid changes, due principally to the increase in rotative speeds, which resulted in greater economy in steam consumption per kilowatt-hour and decreased cost per kilowatt of the entire unit.

It thus became possible to purchase a 22,000-kw. turbo-generator, at 1,500 r.p.m., the steam consumption of which was 12½ lb. per kw.-hr., in place of the 12,000-kw. vertical unit previously planned, which like No. 2 would have had a steam consumption of 15½ lb. per kw.-hr. at 750 r.p.m. In short the capacity of the unit was nearly doubled, the efficiency increased 20% and the cost per kilowatt decreased by about 30%. At the same time the vertical machine would fit into just the space required for the smaller unit. The principal dimensions of the new turbo-generator are as follows: Length overall, 45 ft.; width overall, 19 ft. 2 in.; weight of turbine, 241,000 lb.; weight of generator, 342,000 lb.; total weight, 583,000 lb.

The turbine required a condenser having 35,000 sq.ft. of surface; the condenser shell and tubes weighed, empty, 300,000 lb., and when completely filled with water, 618,000 lb.

The conditions in the station, with regard to condensing tunnels and adjacent apparatus, with the necessary requirements of accessibility and handling, made it necessary to install the condenser immediately under the steam turbine, with its long dimension parallel to the center line of the turbine shaft.

*Designing Engineer, Edison Electric Illuminating Co., 360 Pearl St., Brooklyn, N. Y.

As in the reaction type of turbine the clearances of the blading are very close, it is extremely necessary in planning a foundation for such a unit to make deflection negligible and to reduce the vibration to a minimum. For this purpose mass concrete for the foundation would have been best, but on account of the position of the condenser, the height available from basement floor to the main floor of the turbine room and the clearances necessary, it was found absolutely impossible to provide space for a concrete foundation. Structural steel was naturally the next available material and it was decided to erect a steel platform with very liberal allowances for factor of safety.

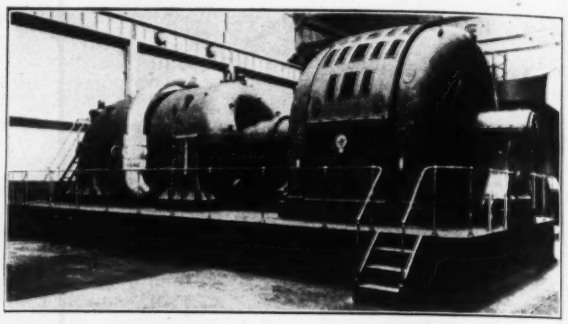


FIG. 1. 22,000-KW. TURBINE ON STEEL PLATFORM CARRIED ON AN INGENIOUS GRILLAGE SYSTEM
Area of unit, 45x19 ft.; weight of turbine, 120 tons; weight of generator, 171 tons

## Condensing Tunnels Pass Through Site

There are two concrete condensing-water tunnels in the station, both of which pass through the area occupied by No. 3 foundation, as shown in Fig. 2. The easterly one consists of a condensing inlet and a discharge tunnel; the other tunnel is for inlet water only and extends some 24 ft. below the surface of the basement floor, hereafter referred to as datum. In the first case the discharge tunnel is over the inlet tunnel, each of them of horseshoe shape 8 ft. wide by 8 ft. deep. This structure was not strong enough to carry the load of the turbine foundation and machine; it therefore was necessary to build the foundation independent of the tunnel structure.

The westerly tunnel, built in 1912, is of reinforced-concrete construction lined with cast-iron plates. It is on good bottom, so that there was no danger in placing some additional weight on the concrete wall.

Referring to the cross-section, shown in Fig. 2, it will be seen that the structural design adopted consists of a pair of cantilever girders resting on a fulcrum of three girders supported by sectional tubular-steel piles filled with concrete. The cantilever girders carry in turn the middle columns of the turbine foundation at one end just beyond the fulcrum, and at the other end the two columns under the steam end of the turbine. The same girders support the condenser, as shown in Fig. 2. The columns supporting the outboard end of the generator are entirely independent of the rest of the foundation, because of its

being impossible to carry the foundation over the discharge-tunnel structure. In this case it was unnecessary to put in additional substructure, as there were good wooden piles in place; furthermore, the soil was sufficient to carry the load of 1½ tons per sq.ft., even had the piles not existed.

### Cutting Out Existing Concrete Foundation

In constructing the foundation the first work to be done was to clear away and cut out all the mass concrete previously installed for No. 3 foundation. The mix was 1:3:5 (gravel).

The method consisted of drilling rows of holes 6 in. apart to the depth desired, using air and steam drills, and then breaking off in shelves with plug and feathers. The concrete, while hard and homogeneous, was very difficult to excavate. Instead of coming away in blocks, it shaled off in 3- or 4-in. layers, and in no case were the pieces as large as one-man stone. This fact rendered the progress of the work very slow, and the cost of excavation ran up to an almost prohibitive figure.

When excavation had been carried on sufficiently to permit piledriving, a machine was brought into the building and set up over the work, and driving began.

### How the Piles Were Tested

When about half of the piles had been driven to the refusal stated, or work had been stopped on them on account of obstruction, it was decided to concrete two piles typical of existing conditions and test them with actual load, so as to be able to judge the necessity of going deeper with the remaining piles.

The piles were designed to carry a load of 20 tons each, and it therefore was desired to load them with at least 30 tons and if no settlement occurred, to continue the loading up to 40 tons. The test rig used is shown in Fig. 3. It will be seen from the drawing that the platform of I-beams on which the load rests is supported on jacks during the loading process. Provision is made to back off the jacks slightly at first for balancing, and afterward back away with 1-in. or more clearance of the blocking to allow for possible settlement. This precaution prevented accident from any shifting of the load.

Readings were taken with a level on an iron rod resting directly on the pilecap and protected from the load pigs by an iron pipe through which it passed freely. To the level rod was secured a piece of white paper on which a fine pencil line was drawn at the start of each loading, so that very accurate readings were possible.

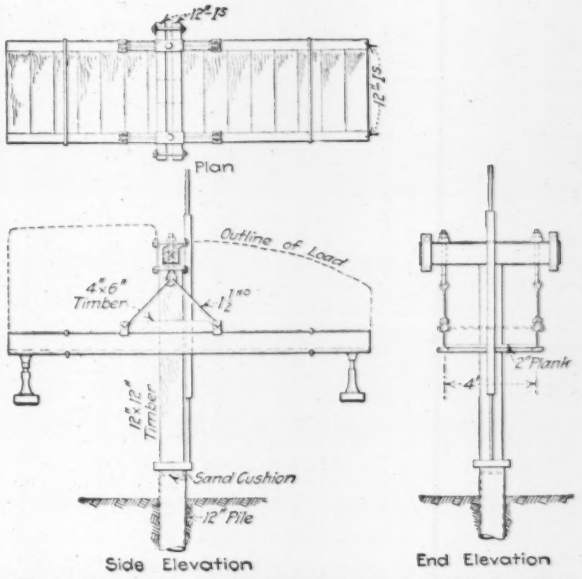



FIG. 3. GENERAL ARRANGEMENT OF RIG FOR TESTING THE PILES

The tables which follow show the results obtained on Piles 11 and 12. No. 11 pile had only one section, 18 ft. long, the driving having been stopped on account of the bottom of the pile turning over against a boulder or other obstruction and preventing the bucket from going through. As this pile was at the outer edge of the footing and extreme difficulty had been experienced in driving it, it represented about the worst condition of any of the piles, and a 25-ton test was considered sufficient.

No. 12 pile, on which the other test was made, was a very satisfactory one. The pile had been driven successfully to a depth of 21 ft. 7 in. below the excavation, or 34 ft. 7 in. below datum, in two sections of 18 and 5 ft. respectively, and the excavated material showed that it was fully 3 ft. into good sand bottom.

After the rig had been shifted, the second test was started at 30 tons. As there was no settlement at all the load was continued to 35 tons and then as settlement was inappreciable, being only $\frac{3}{32}$ in. in 24 hr., the load was continued to 40 tons, resulting in the very small settlement of $\frac{5}{64}$ in. In the interest of experience and because the rig proved so very practical the load was run up to 42½ and 45 tons with the results shown in the table. The test on pile 12 was so satisfactory that it was felt that ample factor of safety was provided, even making allowance for the piles driven to less than the full depth.

The loading material consisted of pigs of cast iron, each weighing about 42 lb. As they were borrowed for the purpose the tests were made at very reasonable cost.

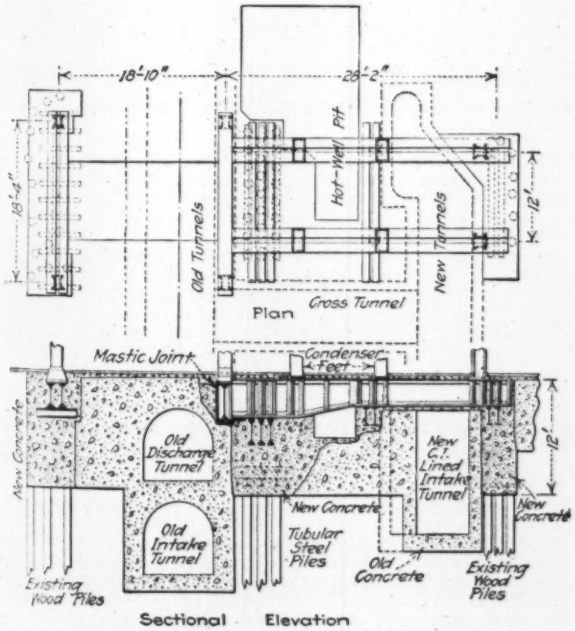
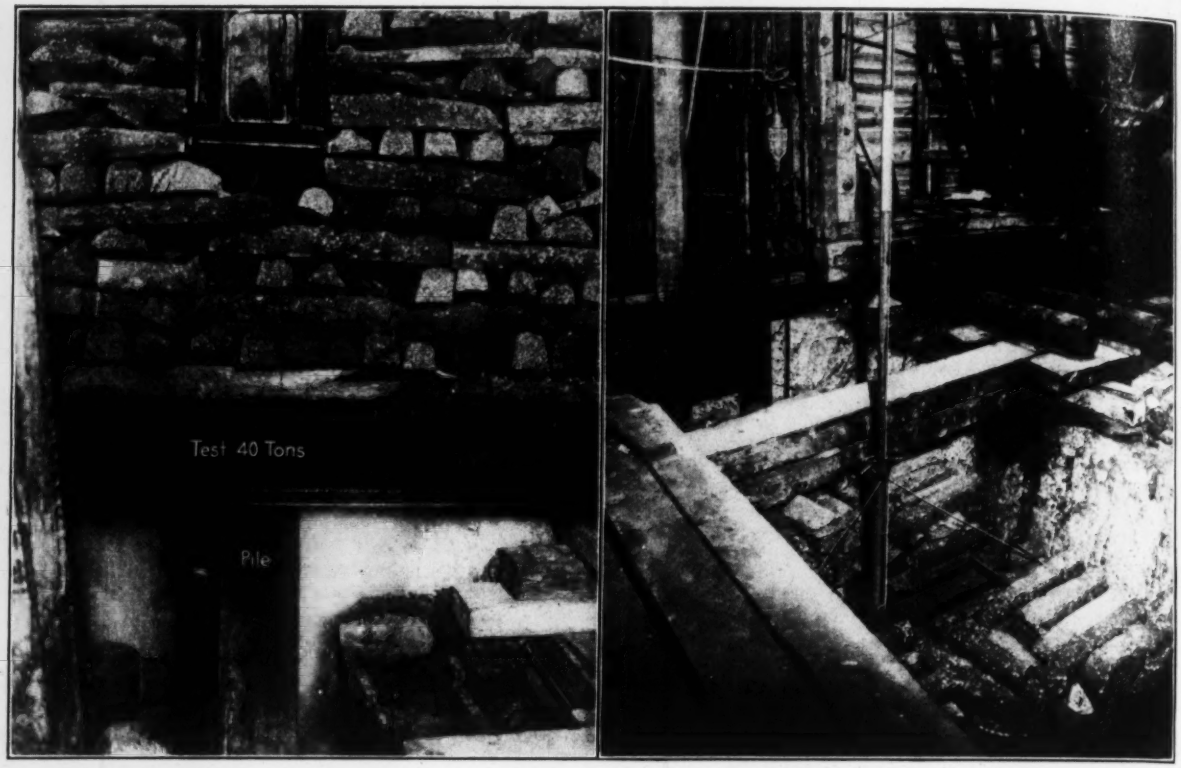



FIG. 2. FOUNDATION SUBSTRUCTURE, SHOWING CANTILEVER GIRDERS AT RIGHT




FIGS. 4 TO 5. (LEFT) METHOD OF SUSPENDING TEST LOAD. (RIGHT) LEVEL ROD WITH WHICH READINGS WERE TAKEN

From this point the placing of the piles was completed expeditiously. As soon as placed, they were dredged out with orange-peel buckets and filled with concrete. As the piles when driven contained water to depths varying with the condition of the soil at the bottom of the pile, the concrete had to be placed with a tremie, precaution

TABLE 1. LOADING TEST, PILE NO. 11

| Date | Load, Tons | Time Load On | Time Load Off | Settlement |
|---|---|---|---|---|
| Oct. 25, 1914 | 25 | 11:00 a.m. | | Down 11/64 in. on cap when first put on, due to compression. |
| Oct. 26, 1914 | 25 | | 11:45 p.m. | Up 3/64 in.—not accounted for. |

TABLE 2. LOADING TEST, PILE NO. 12

| Date | Load, Tons | Time Load On | Time Load Off | Settlement | Remarks |
|---|---|---|---|---|---|
| Oct. 27, 1914 | 30.0 | 9:30 a.m. | 3:10 p.m. | 3/16 in. due to compression of sand cushion under cap when load was first put on. | No settlement thereafter. |
| Oct. 27, 1914 | 35.0 | 4:00 p.m. | 7:30 a.m. | 1/16 in. immediately; 1/32 in. next half hour. | No settlement thereafter. |
| Oct. 28, 1914 | 40.0 | 8:38 a.m. | 4:25 p.m. | 5/64 in. immediately. | No settlement thereafter. |
| Oct. 28, 1914 | 40.5 | 4:49 p.m. | 7:15 a.m. | 1/64 in. immediately. | No settlement thereafter. |
| Oct. 29, 1914 | 45.0 | 8:00 a.m. | 8:00 p.m. | 1/32 in. immediately; 1/32 in. next half-hour; 1/32 in. between 12:30 and 1:30 p.m. | No settlement thereafter to time of load off. |
| Oct. 29, 1914 | .... | ..... | ..... | Up 1/32 in. when load went off. Total, 9/32 in., except initial 3/16 in. | |

first being taken to place about a foot of broken stone in the bottom of the pile, and to tamp it well. After filling, each pile was given about five blows of the driver to settle it in place. Concrete was then placed around the pile tops to a depth of about a foot above them, and a double grillage of I-beams was placed to distribute the load over the piles. The foundation concrete was then brought up to within 2 in. of the bottom of the cantilever grillage girders, in readiness for the steelwork.

It took just one month to place the 23 piles, working 24 hr. a day and 7 days a week.

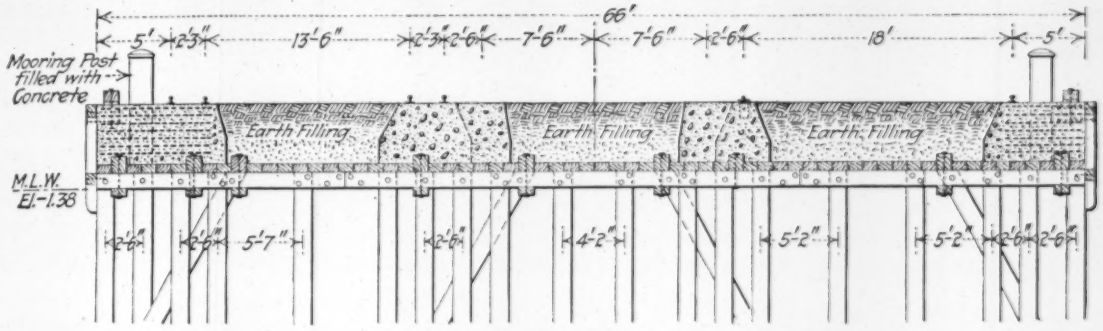
For the foundation for the end columns, on both the east and the west ends, the concrete was placed on a bed of broken stone, which had been tamped solidly to a depth of 1 ft. In one operation the concrete was brought up to a level 1 in. below the bottom of the grillage beams. The steel erection was straight work and presented no unusual difficulties.

**The Seacoast Fortifications** of the United States are described in the annual report of the Chief of Engineers. The present seacoast defenses were planned by the Endicott Board of 1886 and the National Coast Defense Board, of which Hon. Wm. H. Taft was president, which reported in 1906. Fortifications in accordance with the recommendations of these boards have been built at 26 different seaports; in addition, work is in progress on coast defenses at Los Angeles, and batteries are shortly to be built at the entrance to Chesapeake Bay. The total amount appropriated to date for building gun and mortar batteries is a little less than $30,000,000, and a little over $1,000,000 has been appropriated for modernizing some of the older batteries. Additional expenditures for fire control at fortifications, electrical installations, searchlights, preservation and repair of fortifications, seawalls and embankments, and the purchase of sites for fortifications amount to about $15,000,000 additional. The Chief of Engineers says: "When designed and constructed, our seacoast batteries were thoroughly modern and fully adequate for the purpose for which they were intended; but the work of battery construction has in the past few years been allowed to practically cease and has not kept pace with the recent progress in naval development. Unless our fortifications are to become obsolete, changes in any of the elements of naval defense must be met by corresponding changes in seacoast batteries, and these changes must in most cases consist in the construction of absolutely new batteries in new locations. If the guns mounted in the older batteries had been of longer range, the locations of the batteries would in many cases have been different."

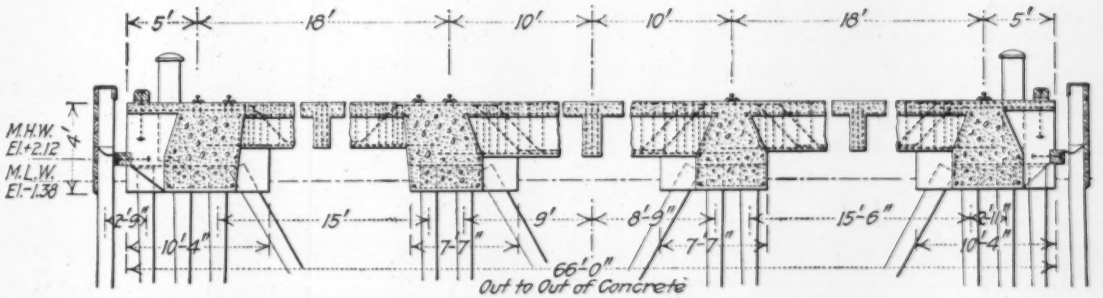
# Reinforced-Concrete Coal Pier for Pennsylvania R.R.

In the construction of its new coal pier at Canton wharves, Baltimore, Md., the Pennsylvania R.R. is utilizing a type of reinforced-concrete pier that showed a notable saving over the timber and earth-fill design first proposed. The pier, which is to be 66 ft. wide and 942 ft. long, is to be equipped with the most modern coal-handling machinery. The usual barney cars propel the coal cars up an incline to a car dumper that empties the cars into a hopper, from which small cars deliver the coal through conveyors into the holds of vessels lying alongside the pier. The plant provides facilities for loading 20,000 tons of coal per day to vessels and is to be approached through a yard with a capacity of 1,000 cars.

As at first designed, the pier consisted of a substructure of timber piles carrying a timber floor on which were to be an earth-fill about 4 ft. deep and four longitudinal concrete stringers to carry the steelwork of the superstructure. A section through this pier is shown in the upper part of the accompanying drawing. As an improvement on this design, the company later adopted the one shown in the lower part of the drawing, which is now being executed.




Proposed Timber and Earth Fill Design




Reinforced-Concrete Design being Built

PROPOSED AND ACCEPTED CROSS-SECTIONS OF COAL PIER FOR PENNSYLVANIA R.R. AT CANTON, MD.

The adopted design is entirely of concrete above low water. It consists of four longitudinal massive concrete stringers carried on timber-pile bents (with inclined piles, as shown) and carrying a reinforced-concrete floor made up of transverse beams 12 ft. 6 in. c. to c., with an integral 10½-in. floor. The superstructure reactions come above the stringers.

To show the actual saving in weight of the reinforced-concrete pier over a timber-floor earth-fill, the quantities in the accompanying tables, taken from the engineers' figures, are of interest.

TABLE 1. WEIGHT OF TIMBER-FLOORED, EARTH-FILLED PIER (FIG. 1)

| | Tons |
|---|---|
| 686,000 ft. b.m. yellow-pine wet timber at 4½ lb. | 1,543 |
| 3,530 cu.yd. concrete at 3,900 lb. | 6,883 |
| Reinforcing steel | 19 |
| 5,630 cu.yd. earth-fill at 3,000 lb. | 8,445 |
| Bolts and cast-iron washers | 34 |
| Total | 16,924 |

TABLE 2. REINFORCED-CONCRETE PIER

| | Tons |
|---|---|
| 5,960 cu.yd. concrete at 3,900 lb. | 11,622 |
| Reinforcing steel | 118 |
| Bolts and washers | 2 |
| 50,000 ft. b.m. timber waling strips, etc., at 4 lb. | 100 |
| Total | 11,842 |

The difference of over 5,000 tons in the weight of a dock is illustrated in the comparative number of piles required—for the timber dock, 4,332; for the reinforced-concrete dock, over 1,300 less piles, or 3,084.

The coal pier is being built by the Pennsylvania R.R., Alexander Shand, Chief Engineer. The Cleveland Dock Engineering Co., Cleveland, Ohio, designed the pier substructure, which is being built by the Arthur McMullen Co., of New York. The dredging has been done by the Maryland Dredging and Contracting Co., of Baltimore, Md., and the superstructure will be furnished by the Mead-Morrison Manufacturing Co., of Chicago.

**Electric Cooking and Heating** are being stimulated by the Potomac Electric Power Co., of Washington, D. C., which has made a radical reduction in the rates covering current used for such purposes. The rates, effective Dec. 25, 1915, are 10c. per kw.-hr. for the first 10 kw.-hr. used each month, then 3c. for the remaining current used during the month. In order to separate the current used for heating and cooking from that which may be used for lighting, a separate meter will be installed for each user, the minimum charge for such meter being $1 per month unless there is a lighting meter on the same premises, in which case the charge is 50c. It is estimated by the company that the charge to the consumer will be practically the same as the charge for gas for corresponding service.

## Oiling Illinois Earth Roads

The economical maintenance of good earth roads in the black-soil district of Illinois is a specially difficult problem, an effective solution of which seems to be the oiling of such roads. Oiled earth roads have been used in Iroquois County with good results, although the effect of the winter is yet to be seen. Several townships last summer oiled an aggregate of about 20 mi. of roads in the heavy black-soil districts, no work being done on the sand roads. In a paper presented at the annual meeting of the Illinois Society of Engineers and Surveyors by Benjamin Jordan, county superintendent of highways, it is stated that even in the wettest weather, when other roads were almost impassable for machines, these oiled roads were practically as good as asphalt paving.

One township used for 8 mi. of road three tank cars of oil (30,441 gal.), costing $829, which was paid for out of the township-road fund. Farmers donated teams to distribute the oil, so that the 8 mi. cost little over $100 per mi. The result is that three townships will vote on a proposition to levy a special tax of 40 to 50c. per $100 for a period of five years, to be expended in oiling the roads. The vote is to be taken in April, so that the people will be able to see how the present oiled roads stand the winter service.

Each of these townships is supplied with tractors, road graders, etc., and one has purchased a 1,000-gal. oil distributor. The plan is to coöperate so that when one township receives a shipment of oil, it will have the use of the entire plant of the three townships for unloading the tank cars rapidly and distributing the oil economically.

In another paper on the same subject, presented at the meeting by B. H. Piepmeier, maintenance engineer of the Illinois State Highway Department, it was pointed out that the main purpose of oiling earth roads is to suppress the dust and aid in maintaining a smooth and waterproof surface. The oiling does not make a permanent road, but the average user of earth roads is willing to bear three or four months of bad roads during the winter if he has assurance of reasonably good roads for the remainder of the year. Some people think that after a road has been oiled systematically for a number of years the surface will become saturated and no further oiling will be necessary. Examination of streets and roads that have been oiled annually for ten years shows that this idea is erroneous, and taxpayers should be warned against such expectations. While a little less oil may be required after several years' treatment, the oiling cannot be eliminated if a good road is to be maintained.

There are, however, many earth roads that can be maintained effectively by the intelligent use of oil and the road drag. It must not be expected that this treatment of a main road will make it satisfactory for heavy traffic. The surface should be properly graded, drained and rolled before oiling, but this work should not be charged to the cost of oiling. It is better to oil on a wet surface than on a dusty surface, but the road should not be so wet that the wheels of the distributor cart will pick up mud.

For the first application ½ gal. of oil per square yard may be required, or ¼ gal. if the road has been oiled before. It is better to apply small quantities frequently than to make one heavy application, and one of the best roads of this kind was treated four times during the summer at the rate of ⅛ gal. per sq.yd. for each treatment. Dragging an oiled road is absolutely necessary for maintaining it in good condition.

A light application of sand is a justifiable expense for a majority of oiled earth roads. It retains the oil and assists in preventing wear and keeping down the dust. It may be applied by shoveling from a wagon or by a distributing apparatus, in the proportion of 1 cu.yd. of sand to each 150 sq.yd. of road.

## Novel Water-Treatment Plant at Excelsior Springs, Mo.

The water-supply of Excelsior Springs, Mo., is obtained from driven wells in bottom lands of the Missouri River at Miltondale, about 7 mi. from the city. The city contains about 5,000 permanent residents and some 3,000 transients per year. There are 720 service connections, all metered.

The water from the wells contains a considerable amount of iron and at times is highly colored and bad tasting. An analysis shows 10.7 p.p.m. of iron, 72.8 parts of carbon dioxide and an alkalinity of 365 parts. The extraordinary amount of $CO_2$ is noticeable in the effect on lead fixtures and plumbing.

To eliminate these objectionable features, the Excelsior Water, Gas and Electric Co. (S. W. Henderson, vice-president and general manager) employed the firm of Black & Veatch, consulting engineers, Kansas City, Mo., to design and supervise the construction of the apparatus described in this article. The plant, which was completed and tested in August, 1915, consists of an aërating tower of unusual design, settling basins, mixing chambers, mechanical filters and a clear-water storage well, from which the water is pumped into the city mains. The capacity of the filter plant is 1,000,000 gal. daily.

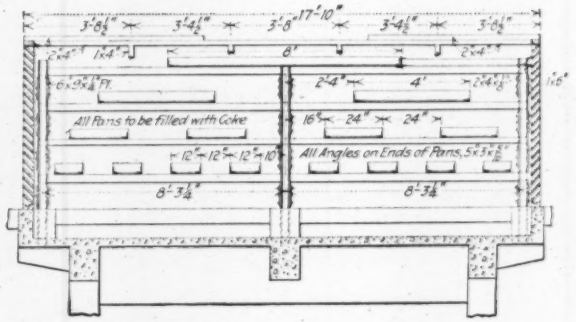



FIG. 1. CROSS-SECTION, SHOWING TYPE OF WATER AERATOR, EXCELSIOR SPRINGS, MO.

The pans are filled with coke. The water flows over the sides of each pan. The vertical drops are 1.9 ft. from the first and 2.3 ft. from the bottom set of pans and the pans are 20 ft. long. The drawing here reproduced is a section of an aërator built at Emporia, Kan., but the cross-sectional dimensions of the two plants are substantially the same, except that the vertical drops in the Emporia plant are 1.25 ft. instead of as stated above for the Excelsior Springs plant

An analysis of the raw water collected on Aug. 30, 1915, by F. M. Veatch was as follows, the quantities being parts per million: Total solids, 527; $SiO_2$, 3; $Fe_2O_3 + Al_2O_3$, 56.6; Ca, 101.2; Mg, 16.82; Na + K, 6.2; Cl, 8; $SO_4$, trace; $HCO_3$, 420.

The water is colorless when first pumped from the ground, but becomes a yellowish red on exposure to the air. The aërator (Figs. 1 and 2) is the most interesting feature of the plant. After being aërated, the water passes through a concrete flume to the mixing chamber, where milk of lime is added. The water then goes to two settling basins (Fig. 3), either one or both of which may be used.

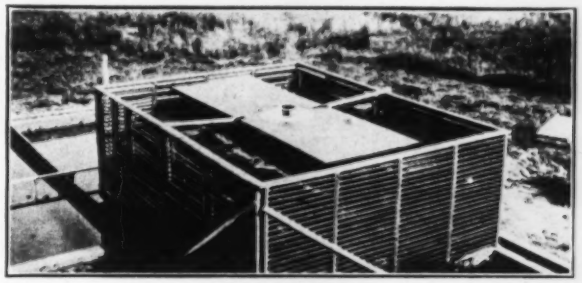


FIG. 2. VIEW OF WATER AERATOR, EXCELSIOR SPRINGS, MO.

### Results of Tests of Operation

The accompanying table gives the results of tests made in the latter part of August by C. C. Young, chemist of the Kansas State Water Survey Laboratory; F. R. Hesser, assistant engineer, Kansas State Board of Health; and F. M. Veatch, of Black & Veatch. The tests were made after the plant had been in operation about 10 days. What follows is based on a report of these tests.

The water had a cloudy yellow appearance after the addition of the lime. All color was removed by the filters. No floc was observable in the basins or on the filters. The raw water had a carbon dioxide content of 85 p.p.m. After passing the aërator this was reduced to 27 p.p.m., and after the lime treatment on reaching the end of basin No. 2 the $CO_2$ had been reduced to 19 p.p.m. In the filtered water it was 11 p.p.m.

There was no increase in bicarbonate alkalinity by the addition of the lime. This was an unusual condition in so in this respect the aërator is giving entire satisfaction. The determinations (in the table) showing 106 and 103% dissolved oxygen are probably due to the presence of algæ, which were growing in the reservoir. The average reduction of carbon dioxide by the aërator was 59% of the total, of which 24% was removed in flowing over the first pan, 6.27% by the second, 3.9% by the third, 16.7% by the fourth pan and 8.1% by the fall into the collector.

Under ordinary weather conditions this leaves about 30 p.p.m. of $CO_2$ in the water, which is sufficient to prevent a large part of the iron from precipitating. At the beginning of the tests sufficient lime was not added to neutralize the $CO_2$, which accounts for the fact that

REDUCTION OF IRON AND CARBON DIOXIDE BY WATER-TREATMENT PLANT, EXCELSIOR SPRINGS, MO.

| | Parts per Million | | | | | | | | | | | Lime Used | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Test For | Raw Water | Pan No. 1 | Pan No. 2 | Pan No. 3 | Pan No. 4 | Collector | Basin No. 1 Mid-dle | Basin No. 2, Mid-dle | Basin No. 1, Second Bay | Basin No. 1, Fifth Bay | On Filter | Filtered | % Sol. | Gr. per Gal. | Gal. Filtered per Min. | Remarks |
| | 8 | .. | .. | .. | .. | .. | 2.5 | ... | ... | ... | 2.0 | 0.2 | 1 | 1.1 | 512 | Both basins in use |
| | 12 | .. | .. | .. | .. | .. | 2.5 | ... | ... | ... | 1.3 | 0.3 | 1 | 1.1 | 512 | Normal operation |
| | 16 | .. | .. | .. | .. | 12 | ... | 3.5 | ... | ... | 3.5 | 0.2 | 1 | 1.1 | 512 | |
| Total Iron | 10 | .. | .. | .. | .. | 9 | ... | ... | ... | ... | 0.2 | 0.1 | 1 | 3.8 | 512 | |
| | 10 | .. | .. | .. | .. | 10 | ... | ... | 0.2 | 0.2 | 0.2 | 0.1 | 2 | 6.9 | 512 | Baffled basin used |
| | 9 | .. | .. | .. | .. | 9 | ... | ... | 0.2 | 0.2 | 0.2 | 0.1 | 3 | 7.9 | 512 | Second basin cut out |
| | 10 | .. | .. | .. | .. | 9 | ... | ... | ... | 0.2 | 0.2 | 0.1 | 3 | 7.9 | 512 | |
| Dissolved Oxygen | 0 | 23 | 25 | 49 | 78 | .. | 106 | 103 | ... | ... | ..... | 9.3 | 1 | 1.1 | ... | Figures represent % saturation for temperature of collection |
| | 0 | 26 | 47 | 56 | 68 | 86 | 80 | 80 | ... | ... | ..... | 9.3 | .. | 1.1 | ... | |
| | 0 | 25 | 48 | 63 | 62 | 88 | 80 | 80 | ... | ... | ..... | 8.6 | .. | ... | ... | |
| | 86 | 59 | 51 | 52 | 35 | 27 | 21 | 19 | ... | ... | 19 | 11 | 1 | 1.1 | 512 | Normal operation |
| Carbon Dioxide | 74 | 55 | 54 | 55 | 36 | 30 | ... | ... | 8 | ... | 11 | 13 | 2 | 3.8 | 512 | One basin in use; non-baffled basin cut out |
| | 80 | 60 | 60 | 56 | 45 | 46 | ... | ... | 10 | ... | 5 | ..... | 2 | 6.9 | 512 | |
| | 63 | 51 | 42 | 36 | 29 | 21 | ... | ... | 4 | ... | 2 | 2 | 3 | 7.9 | 512 | |
| | 61 | 52 | 47 | 41 | 33 | 25 | ... | ... | 2 | ... | 1.5 | 2 | 3 | 7.9 | 512 | |
| | 356 | .. | .. | .. | .. | .. | ... | ... | ... | ... | 356 | 354 | .. | 1.1 | 512 | Normal operation |
| | 360 | .. | .. | .. | .. | .. | ... | ... | ... | ... | 362 | 358 | .. | 1.1 | 512 | |
| Bicarbonate Alkalinity | 366 | .. | .. | .. | .. | .. | ... | ... | ... | ... | 346 | ..... | .. | 3.8 | 512 | |
| | 364 | .. | .. | .. | .. | .. | ... | ... | ... | ... | ..... | 350 | .. | 6.9 | 512 | |
| | 374 | .. | .. | .. | .. | .. | ... | ... | ... | ... | 304 | 300 | .. | 6.9 | 512 | |
| | 372 | .. | .. | .. | .. | .. | ... | ... | ... | ... | 276 | 272 | .. | 7.9 | 512 | |

view of the fact that as much as 7.9 gr. of lime per gallon was used in some of the tests. The normal carbonates ($CO_3$) did not appear, and the bicarbonate alkalinity was reduced 100 p.p.m. while the 23.5 p.p.m. of $CO_2$ was being neutralized. Apparently the series of reactions $2\ CO_3 + Ca(OH)_2 = Ca(HCO_3)_2$ and $Ca(HCO_3)_2 + Ca(OH)_2 = 2\ CaCO_3 + H_2O$ was not followed at any time during the tests.

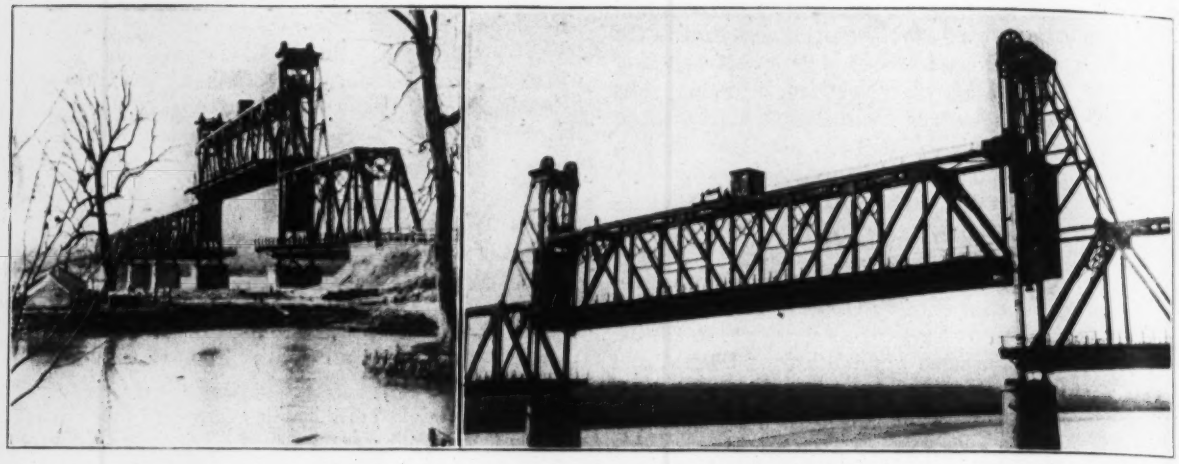
### Action of the Aërating Device

The raw water contained approximately 10 p.p.m. of iron, which after the addition of the lime was reduced to 2 p.p.m. and after filtering to 0.2 p.p.m. The total amount of iron reduction guaranteed was 0.5 p.p.m. The iron is in the water in solution in an unoxidized state and will not precipitate until oxidized and until the greater part of the carbon dioxide has been removed. The twofold object of the aërator is therefore to oxidize the iron and to allow the carbon dioxide to escape.

Under ordinary climatic conditions the water becomes saturated with oxygen by the time it reaches the collector, some of the iron was carried over into the filters. After the preliminary tests were made, the second basin was cut out to increase the rate of flow, and the amount of lime was increased.

The effects of increasing the lime from 1.1 to 3 gr. per gal. were that the flocculent precipitate of iron hydroxide and calcium carbonate appeared at once in the mixing chambers; the deposition of the precipitate took place largely in the first bay of the settling basin, and at the end of 5 hr. the remainder of the basin was practically clear and the water on the filters had only a very slight turbidity. The carbon dioxide in the water on the filters was reduced to 11 p.p.m. and the iron to 0.2 p.p.m. The filtered water contained less than 0.1 p.p.m. of iron.

The tests were continued with 6 and with 7.8 gr. of lime per gallon, in the hope of effecting a complete removal of the $CO_2$, but without success. In accordance with the recommendation of Mr. Young the aërator has since been operated with the first three pans filled with Connellsville egg coke. The lime dose is 2.6 gr. per gal.



FIGS. 1 AND 2. LIFT SPAN HUNG BY CHAINS; ARKANSAS RIVER BRIDGE, PINE BLUFF, ARK.
By shifting towers any span can be made the lift span to suit channel location

# Pine Bluff Vertical-Lift Span Suspended by Chains

*SYNOPSIS—Chains used in place of wire ropes to suspend 239-ft. vertical-lift span across Arkansas River. "Folding" counterweight to balance the chains is another new feature. Various details of novel design shown by drawings.*

Hung by chains instead of by ropes, the lift span of a new county bridge across the Arkansas River at Pine Bluff, Ark., represents a new departure in draw-bridges. Various other features of the lift span are original. The engineers, Hedrick & Cochrane, of Kansas City, boldly disregarded precedent in many ways, not merely in the machinery, but also in the structure. The two wagonways, for example, are placed on long brackets outside the trusses, the deck between trusses being used for electric-railway tracks.

The main low-water width of the river is nearly 2,000 ft. The channel may shift to any position in this distance, as happened at the near-by Cotton Belt bridge, which now has two swing-spans, one near each bank, the channel having shifted from one bank to the other. To provide for such a contingency the county bridge is so arranged that any one of its spans may be made the drawspan. The use of a vertical-lift span enabled this plan to be carried out easily; lift mechanism and tower are so built that they can be shifted to any one of the channel spans, and thus any span—six are of equal span-length—may be converted into the lift span. This expedient has been used before and is known to involve but little extra cost.

The lift span is hung from steel chains instead of from ropes. Also, instead of a lifting cable a lifting chain running over sprockets is used.

The great weight of the counterweight chains making it important to compensate for change of balance due to shifting of the chain weight as the span rises, an expanding counterweight was used as an auxiliary to the main counterweight. It is picked up by the main counterweight as the span goes down, and released as the span goes up.

The six channel spans are each 239 ft. 4 in. long, center to center of end pins. These, as well as a north-approach span of 149 ft. 7 in., are of full riveted type, with 29 ft. 11 in. uniform panel length. The channel spans allow for a clear opening of 225 ft. The one adjoining the short span is the present lift span.

The piers were all sunk by pneumatic caissons to 70 ft. below low water, bringing up on sand and coarse gravel.

### Bronze-Bushed Pin-and-Link Suspension Chains

In the cable counterweight arrangement used in other bridges, a series of wire ropes at each of the four corners of the span is attached in successive pairs of ropes by means of sockets to a number of superimposed equalizer levers, which gradually bring the pull of a number of ropes to a single point of attachment. By using link chains in the present bridge the complication of multiple equalizers is avoided.

The chains consist alternately of four and five links, all of them steel forgings, joined by 3½-in. pins (Fig. 4). Provision for the turning of links on pins was made in alternate links only, the intermediate links being made narrow (with higher pin pressure) to save width of chain and therefore length of pin. The wide links, which turn on the pins, are fitted with phosphor-bronze bushings to reduce friction (see Fig. 4); the bearing pressure is 6,000 lb. per sq.in. projected area. The nonrotatable links are narrow and give a pin pressure nearly three times as great. To hold the pins from turning in these links there is a 1¼-in. diametral hole at each end of the pin just outside the outer link, and a keeper or key bar passed through this hole is bolted to the link by a ⅝-in. bolt.

No provision was made for lubrication. It was thought that, the motion being so slow and so infrequent, lubrication would not be necessary.

When the span was put in operation, many of the key bars bent and some of them broke, showing that the wide phosphor-bronze bearings gripped the pin more tightly

than the thin unbushed "fixed" links. Victor H. Cochrane, of the consulting engineers, says that the cause has not been definitely determined, but that it is believed that the bronze bushings had been machined to a tighter fit on the pin than the pin links. A contributing factor in the trouble may have been temperature expansion, as most of the trouble developed on the approach of warm weather; the greater expansion of bronze than of steel may have helped to cause the bushings to seize on the pin.

For temporary repair, the bent or broken key bars were removed and simple bolts put in their places to act as cotters and keep the links on the pins, irrespective of which set of links rotated. Subsequently, harder key bars were put in than the original medium-steel keys.

The main counterweights of the bridge are of concrete—precast blocks supported in a steel frame. These counterweights are detailed in such a way that they can be taken apart and built together easily when changing to some other span as lift span.

The shifting weight of the counterweight chains is balanced by a set of expanding counterweights (Fig. 4) made of cast-iron disks hung in a loose string from the top of the tower in such arrangement that the main counterweight picks the weights up one by one as the span goes down, and thus constantly increases the counterweighting as the amount of chain on the channel side of the tower increases in the descent of the span. The disks are each 3 ft. 5 in. in diameter and 2¼ in. thick, hung by two 1¾-in. bolts passing through lugs at diametrically opposite points. Successive disks have their suspension bolts in successively different angular position, so that the bolts do not interfere when the counterweight folds up. In the expanded position the disks are 1 ft. apart in the clear. The engineers had in mind the possibility of producing excess compensation in this folding arrangement, by making the disks slightly heavier; such overbalance would help in starting the span from rest and in checking it at the end of its travel. This arrangement was not used, however, but the expanding counterweight was proportioned to balance the chain weight exactly.

Two vertically arranged sprocket chains along each tower post, eight in all, fitted with equalizers and having adjustment to regulate the tension, furnish the means for moving the span up or down (Fig. 7). Just above the top chord of the lift span each pair of sprocket chains is

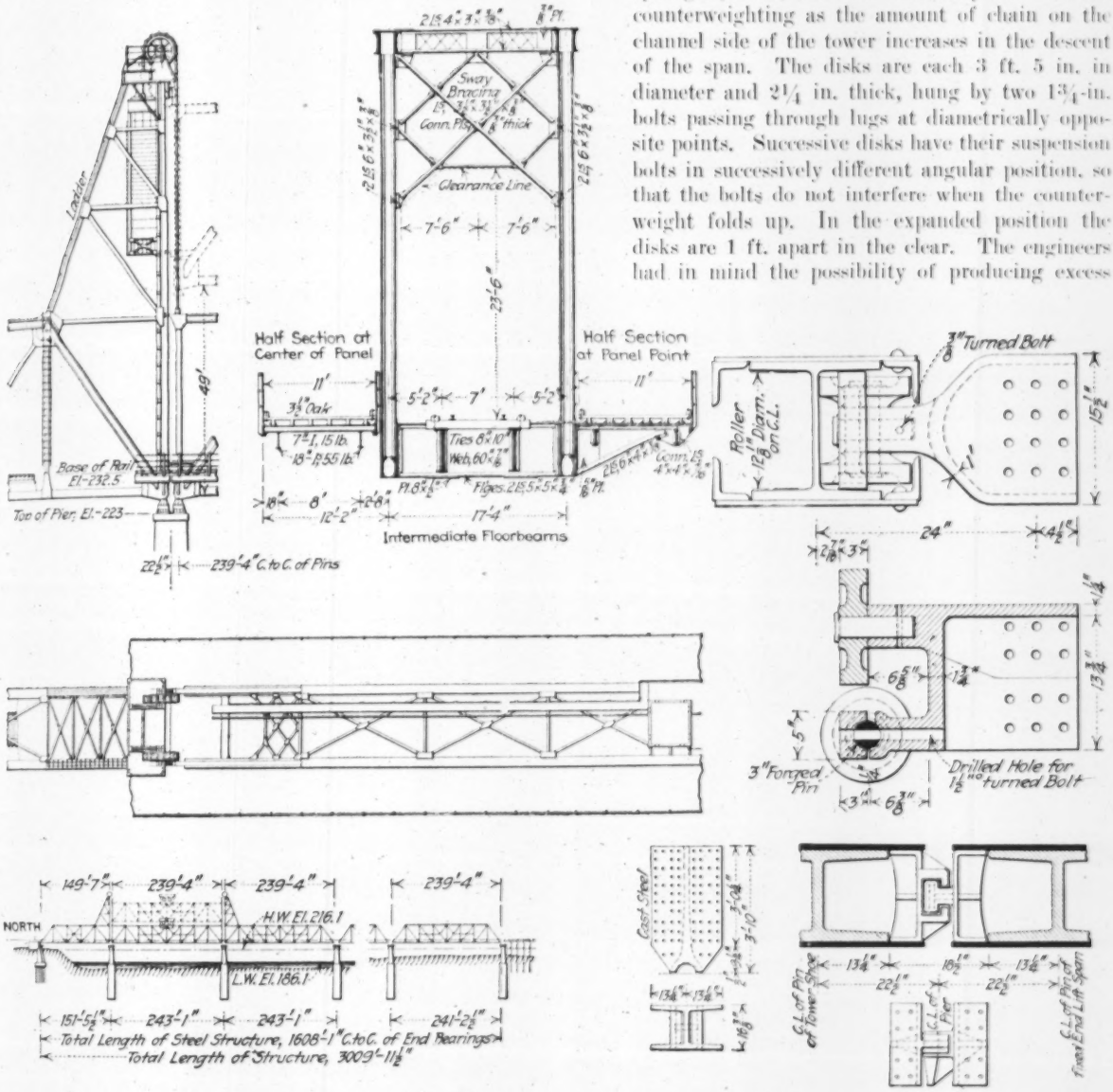



FIG. 3. PINE BLUFF BRIDGE OVER ARKANSAS RIVER

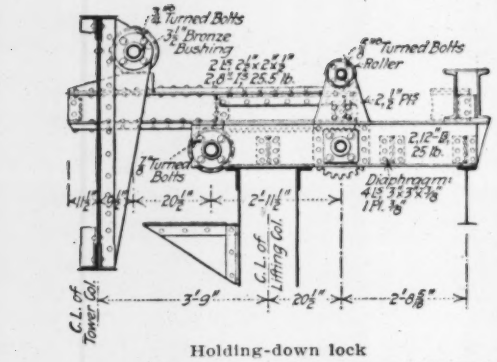
General elevation and section; bearing casting at end pin; guide rollers and traction lock of lift span

looped back toward midspan between two pairs of idler sprockets and around driving sprockets. The latter are operated by a motor through gearing. One motor at each end of the span drives the two sets of sprockets. A longitudinal shaft connecting the gearing at the two ends of the span insures equality of motion at the two ends. It also provides against failure of one of the motors, each motor being capable of operating the span alone. The hand-operating capstan is attached to the longitudinal shaft at midspan.

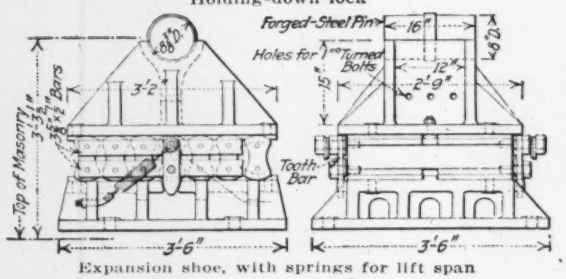
The motors are 1,200-volt d.-c. motors, operated by 600-volt current. It was intended to operate them from the current of the proposed electric railway, but as the railway was not built they had to be supplied by a 2,300-volt a.-c. transmission line. The only available motor-generator set converted to 600 volts, and the motors are operated at this pressure, with no particular difficulty. The rated power at 1,200 volts is 40 hp. per motor.

### Wedge Locking Bars at Ends of Span

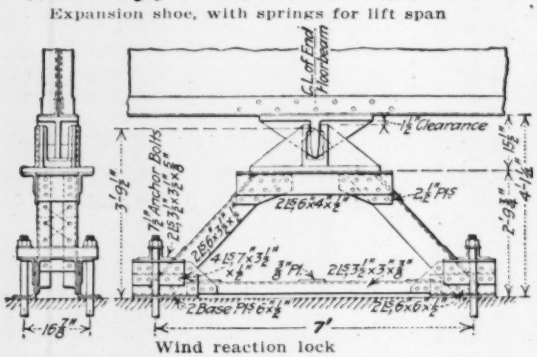
A detail in Fig. 4 shows the end locks. These are long bolts moved by rack and pinion through a small motor. The upper side of the locking bolt is beveled t



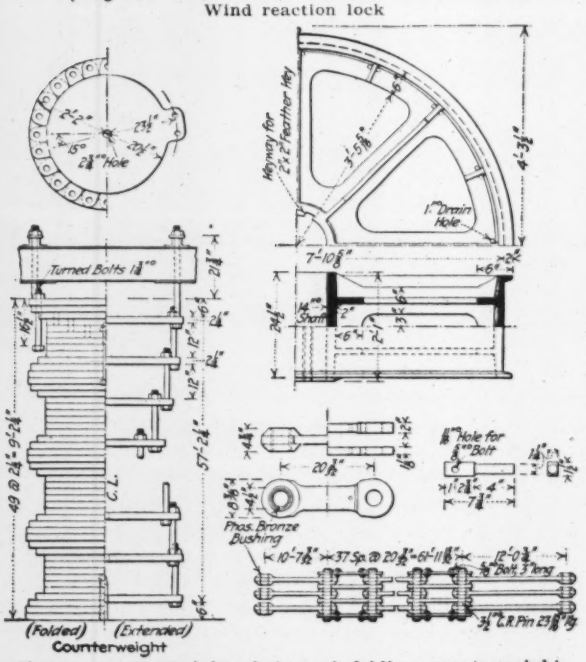

Holding-down lock



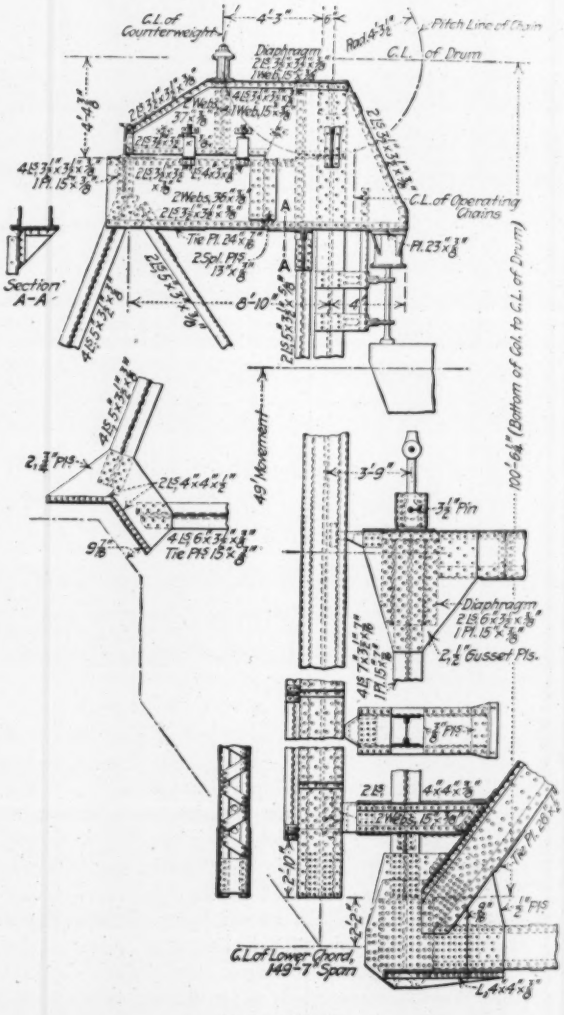

Expansion shoe, with springs for lift span




Wind reaction lock




Sheave, counterweight chain and folding counterweight for chains




Tower details

FIG. 4. IMPORTANT DETAILS OF PINE BLUFF BRIDGE LIFT SPAN

in. in 18 in., so as to force the span down to seat in case it should not be down.

The lift span is guided by the web and flanges of the 14-in. I-section of the tower posts. Guide rollers (Fig. 3) attached to steel castings on the longitudinal strut that continues the line of the top chord from the hip to the counterweight hanger run in the post channel-ways. At one end of the span there are two longitudinal rollers 10 in. in diameter and a transverse roller 12½ in. in diameter, the latter being beveled to suit the bevel of the flange of the I-section. At this end therefore the span is guided both longitudinally and transversely. At the opposite end the two longitudinal rollers are omitted, so that the span is free to expand and contract longitudinally.

These rollers, however, do not have to transfer the traction stresses of braked trains. These are absorbed by a tongue-and-notch engagement of castings riveted between the gusset plates at the fixed end (Fig. 3).

For lateral centering and to transfer the wind stress there is at each end of the span a casting attached to the end floorbeam of the lift span, so arranged that a tongue on this casting engages a recess in another casting carried on the steel frame attached to the pier (see Fig. 4).

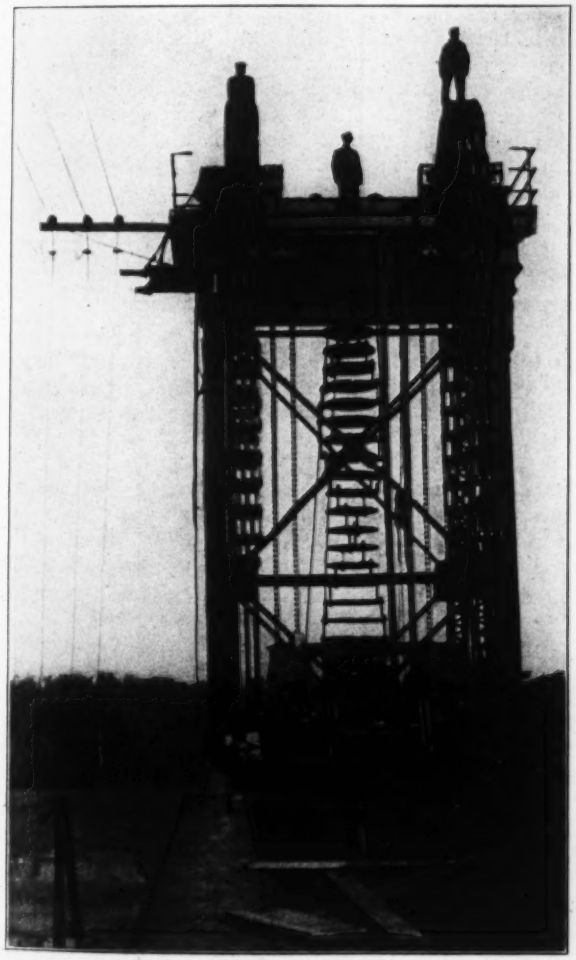


FIG. 5. LIFTING CHAINS AND EXPANDING COUNTERWEIGHT, FROM MIDDLE OF LIFT SPAN

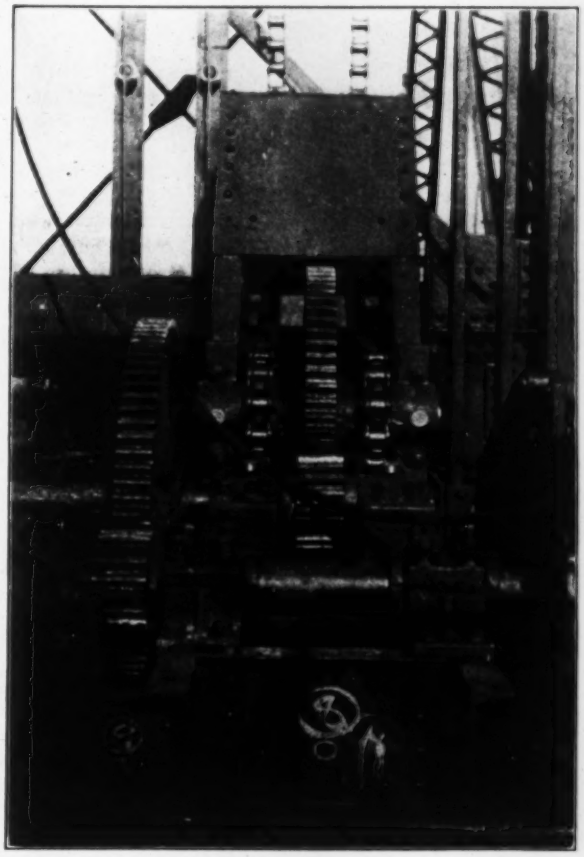


FIG. 6. DRIVE CHAINS FOR RAISING LIFT SPAN

The rail joints are made by a casting each on span and pier. The castings on the lift span have a ¼-in. side bevel to prevent striking the fixed rail as the span comes down to seat.

The shoes of all the spans are designed to be capable of doing service as shoes for the towers. The shoes on which the towers are seated are fixed; that is, they have no rollers. All the expansion bearings of the fixed spans, however, have rollers. In consequence, castings are provided to be inserted in place of the rollers to convert an expansion bearing into a fixed bearing when it is to carry the tower.

As the rollers under the lift span are free when the span is open, they can become displaced. To prevent this, they are held in vertical position by a pair of springs attached to a tongue bar of the middle roller as shown in Fig. 4. The lateral-centering bar used on the other shoes, engaging grooves at midlength of the rollers, is omitted in the case of the lift span. In all other respects the lift-span shoes are the same as used with the fixed spans.

The upper pin casting of each span is a deep casting riveted between the webs of the end post, adapted to take and transfer the large load from the tower. This casting is shown in Fig. 3, and its position in Fig. 4. Since the tower leg must carry the weight of half the span plus its share of the counterweight, or double the normal load, the shoes are a little more than twice as heavy as they would be in ordinary bridge service.

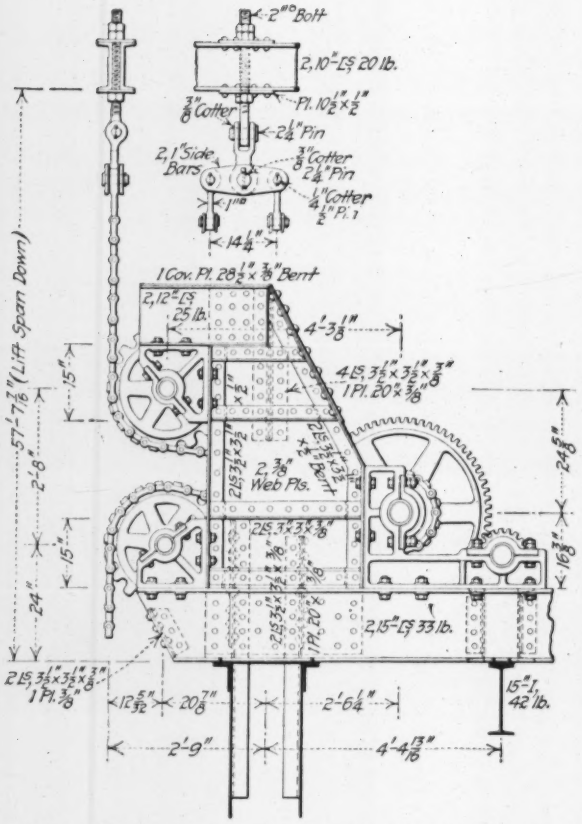



FIG. 7. DRIVE CHAIN AND GEARING OF PINE BLUFF LIFT SPAN

The bridge was built by the Missouri Valley Bridge and Iron Co., Leavenworth, Kan., for whom the American Bridge Co. furnished the steel. Hedrick & Cochrane, the consulting engineers, were represented in the field by C. S. Reagan.

## A New Concrete Pile Driven from the Point

By C. S. Howell*

Concrete piles of a new type are being driven in building the foundations of the Manhattan State Hospital power house on Ward's Island, New York City. In driving these 105 precast piles to bedrock at depths varying from 12 to 20 ft. below the footings, a follower consisting of a pair of driving frames resting on the shoulder of the iron point of the pile and extending beyond its head was placed around the pile, as shown in Fig. 1.

The piles are spaced on average centers of 3 ft. The soil is a very compact sandy clay, and many boulders and smaller stones exist at various depths, mostly just above bedrock. Footing trenches will be excavated after the piles have been driven, which will involve from 7 to 10 ft. of additional penetration above the head of each pile.

The piles are 16 in. square, with corners beveled 1 in. to prevent chipping in handling. They are made of 1:2:4 concrete, ¾-in. crushed traprock and graded sand being used for aggregates. The mix is made wet to pour into the forms. Casting is done in stacks three tiers in height. Each pile has four twisted steel rods extending the full length, and 3 ft. beyond the head for anchorage in the footings. The rods are ⅝- or ¾-in., according to the length of the pile. No tie wires are used. The method of casting provides for holding the rods in correct position while pouring.

After casting, the piles are left to season for four weeks before driving. Forms are removed in 48 hr. and reused. Casting of these piles was carried on from Nov. 22, 1915, to the latter part of December. After seasoning for two weeks, the piles were handled with the same freedom that would attend the handling of timber piles.

The most distinctive feature about these piles is the cast-iron point, which is 19 in. square and weighs over 300 lb. It is hollow, except for four diagonal walls, or webs, through which the reinforcing bars are drawn and bent back upon themselves, the bars being heated for this purpose in a forge on the work. The point has at the top a collar that fits into one end of the casting form, and the rods are drawn through the opposite end of the form, having first been anchored in the point.

The method of driving involves a wide departure from existing practice. Reference to the accompanying drawings will illustrate this in detail. While the practice of driving by means of an inclosing steel casing is not new, it will be noticed that this method uses two frames, on opposite sides of the pile and resting on the iron point. These are of channel shape and have at the bottom two projections that fit into slots cast on corresponding opposite sides of the point, to hold them in position upon the extended part of the point while driving. Earth pressure also serves the same purpose as these projections and their corresponding slots, after sufficient penetration is obtained. The object of using two driving frames is to overcome resistance against pulling them from the

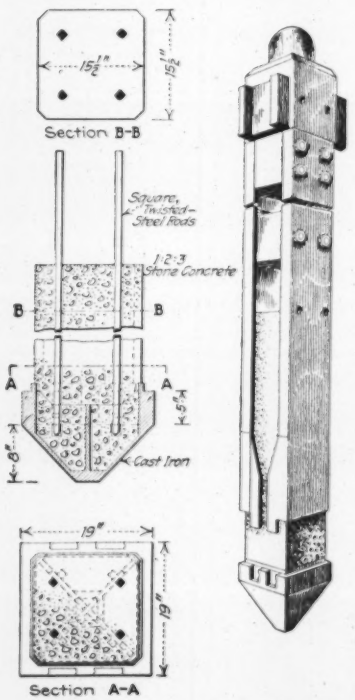



FIG. 1. THE "GIANT" CONCRETE PILE WITH DRIVING CASING ATTACHED, AND DESIGN DETAILS OF THE PILE

*Chief Engineer and General Manager, Giant Concrete Pile Co., 37 East 28th St., New York City.

ground after the desired penetration is reached and to provide sufficient earth friction against the pile to prevent its being disturbed when the casing is pulled.

By this method of driving, the reinforcing rods are made to transmit the resulting frictional stress to the concrete and to distribute this stress over the full length of the pile. A 5,000-lb. steam hammer (total weight, 10,000 lb.), with normal stroke of 42 in., is used to drive these piles. The weight of the hammer rests directly upon a steel-bound oak block held in a steel casting, to which the two driving frames are rigidly secured at the top. The oak block extends through the hammer base and is capped with a steel plate, to receive the hammer.

FIG. 2. A NUMBER OF REINFORCED-CONCRETE PILES WITH CAST-IRON HEADS ATTACHED, READY FOR DRIVING

The driving frames are considerably longer than the pile, the space above the pile head varying according to the length of the pile. At the head of the pile a wood block may or may not be bolted between the frames (bored for the reinforcement), to take a part of the stress due to earth friction on the two partly exposed sides of the pile, when the depth of penetration develops a pull in the rods in excess of their safe tensile strength. Thus far such a block has not been used on the hospital work. Since the point is driven by the frames and inasmuch as these frames also relieve the pile of all earth friction, except as already noted, the pile is actually made to occupy the displacement of the point, as penetration proceeds, by means of the rods.

The first of these piles was driven on Dec. 30, 1915, to a depth of 17 ft., when refusal under the last 30 blows indicated bedrock and closely checked the preliminary borings. The pile was driven in 13 min., 290 blows being required. The rapid penetration, though it proceeded in constantly decreasing increments, was due to the weight of the driving apparatus, there being over 22,000 lb. in movement. The second pile was driven 6 in. deeper and required 336 blows and 15 min. The piles were not damaged in any manner in driving, there being no chipping or indication of cracks.

## Municipal Street Sprinkling

The charter of the City of Rochester, Minn., was amended early in 1915 to allow the city to do its own street sprinkling, charging to the frontage the exact cost of the work. The results were described by J. C. Utton, City Engineer, in a paper presented at the annual meeting of the Minnesota Surveyors and Engineers Society at St. Paul.

The district comprised 5¾ mi. of paved streets and 4 mi. of unpaved streets. The city had four 600-gal. sprinkling wagons, and the district was divided into four sections to equalize the travel of the teams, which averaged about 14½ mi. daily. The paved streets were sprinkled four times daily and twice on Sunday, and the others were sprinkled twice daily and once on Sunday. The teams were hired at $110 per month, and water was supplied by the local company at 1c. per 100 gal. of water, each tank being equipped with a meter.

The sprinkling season covered six months. Under the charter the cost of sprinkling each street has to be kept separately, so that special report blanks were necessary, entailing a large amount of bookkeeping. Daily reports showed the amount of water used on the street and the time spent. Monthly reports were made up from these daily records, showing for each street the total amount of water and the labor cost for hauling and sprinkling.

The cost estimate for the season was based on 100 sq.ft. of surface, including street intersections. The entire cost, including intersections, was assessed against the abutting property on the frontage basis. Corner property was assessed in full for each frontage. The average cost to the property owners was 0.2 to 0.57c. per front foot on the paved streets and 0.2 to 0.51c. on the unpaved streets. The cost under city force account in 1915 was $2,985. In 1914, under the contract system, the cost was $3,086. This latter was for less frontage and yardage sprinkled, while the city furnished the wagons and paid half the water cost.